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Final Report

ASSESSING THE COST/BENEFITS OF EMPLOYING THICKER BRIDGE DECKS IN ALABAMA

Submitted to

Highway Research Center
Auburn University

Prepared by

G. Ed Ramey
Daniel M. Balmer

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Final Report

on

Highway Research Center Research Project

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ABSTRACT

Alabama employs thinner decks on their highway bridges than all other states in the U.S. Up until about two years ago, ALDOT deck depths ranged from 6 1/4" to 7 3/4 ", with the depth depending primarily on the girder spacing. About two years ago, the minimum depth was increased to 7" and thus ALDOT's deck depths now range from 7" to 7 3/4". While this increase in minimum value is viewed by most parties as being a step in the right direction, this upgrade does not affect bridge decks with the very common girder spacings of 8 feet or more. Also, even with the upgrade, ALDOT still employees the thinnest decks in the country.

Thicker bridge decks are stiffer, stronger and should provide a longer service life; however, they cost more and require a stronger and more costly support girder system. Thus, the cost/benefits of using thicker decks is not clear and this was the impetus for this study.

The results of performing a parameter sensitivity study, talking with ALDOT bridge design engineers and bridge contractors, and a survey of other state DOTs all point to increasing bridge deck thickness to a minimum of 8 in. All of the parameters investigate in the parameter sensitivity study, with the exception of two (deck unit weight and initial cost), support increasing Alabama's bridge deck thicknesses to 8". An increase in deck thickness from 7" to 8" will cause the deck unit weight to increase 6-12 psf depending on whether the concrete is added to the top or underside of the deck. This will increase the cost of the bridge deck around \$0.20ft^2, and would translate into an increase in deck/bridge initial cost of around 2%-3%. However, increasing the deck thickness from 7" to 8" would also increase the deck service life (10-50%) which would reduce the life cycle cost of the deck/bridge.

From the information developed and collected in this study, it is recommended that bridge deck thicknesses in Alabama be increased from a minimum of 7" to a minimum of 8".

This will increase the durability and longevity of the bridge decks and in turn lower the life cycle cost of decks/bridges. Additionally, a deck thickness of 8" should become the standard and used throughout the state except for unique circumstances. This will improve construction quality and lower cost by simplifying the construction process. Also the deck top bar cover should be increase from 2" to 2 ½" to allow for construction errors and errors in overestimating the deck DL deflection. This will help insure a minimum cover of 2". These changes will help improve the quality of bridge decks in Alabama with minimal increase in initial cost and an anticipated reduction in life cycle cost.

ACKNOWLEDGMENTS

This research was supported by the Highway Research Center (HRC) and Civil Engineering Department at Auburn University. The PI is grateful to these sponsors for their support of this work. The PI is also grateful to Mr. Keith Mims, President of Alabama Bridge Builders, Inc., Mr. Tim McInnis, Vice President of McInnis Corporation, and Mr. Charles Davis, Jr., Chief Engineer for Scott Bridge Company, Inc., for meeting with him to discuss the effects of increasing the thickness of bridge decks from construction and cost perspectives. Lastly, the PI appreciates the cooperation and assistance of the state DOT's via their timely and conscientious response to our mail survey questionnaire.

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1. INTRODUCTION

1.1 Statement of Problem

The Alabama Department of Transportation (ALDOT) employs thinner decks on their highway bridges than almost all other states and countries. Up until about two years ago, ALDOT deck depths ranged from 6 ¼' to 7 ¾", with the depth depending primarily on the girder spacing. About two years ago, the minimum depth was increased to 7" and thus ALDOT's decks now range from 7" to 7 3/4". While this increase in minimum value is viewed by most parties as being a step in the right direction, this upgrade does not affect bridge decks with the very common girder spacings of 8 feet or more. Also, even with the upgrade, ALDOT still employs thinner decks than most other highway agencies. Thicker bridges are stiffer, stronger and should provide a longer service life; however, they cost more and require a stronger and more costly support girder and substructure system. Thus, the cost/benefits of using thicker decks is not clear and should be investigated. This is the impetus for this research.

1.2 Project Objectives

The purpose of this research is to determine the "best" bridge deck thickness (es) for Alabama based on life-cycle cost/benefit estimates. The specific objectives of this research toward that end are as follows:

1. Identify via mail survey minimum deck thicknesses and typical deck thickness vs. girder spacing used by other DOTs in the U.S.
2. Identify the primary deck and support girder parameters affecting the design, construction and costs of bridge decks and superstructures.
3. Identify the primary advantages/disadvantages of using a standard deck thickness (except for unique circumstances) such as 8".
4. Perform a parameter sensitivity study to determine the cost/benefit of using thicker decks, and to identify "the best" deck thickness (es) for Alabama.

1.3 Scope of Work

This study assesses the sensitivity of bridge deck primary design, performance and cost parameters to deck thickness. Deck serviceability/durability was considered along with deck strength in the assessments, which were based on theoretical analyses, experiences of other DOTs, and discussions with ALDOT bridge engineers and Alabama bridge building contractors. No laboratory or field-testing of decks were conducted as part of this investigation, and only CIP reinforced concrete decks were considered.

1.4 Work Plan

It is possible to theoretically examine the variables which influence the deck thickness, and also to examine the effect that deck thickness has not deck performance, durability/longevity, costs and other major components of a bridge, i.e., the bridge support girders, bents and foundations. This is the purpose and objective of this research. A brief chronological outline of the work performed to accomplish the project objectives is presented below.

1. Perform background reading on design of bridge decks, and a literature review of bridge decks and factors affecting deck thickness decisions.

2. Meet with ALDOT bridge design engineers to (1) review AASHTO and ALDOT bridge deck and support girder design requirements and procedures, (2) to identify the primary effects of increasing deck thickness on the design of the other bridge components along with the associated cost increases, and (3) to discuss the advantages/disadvantages, from a design perspective, of using a standard deck thickness of 8".

3. Prepare a mail survey questionnaire to gather information on deck thicknesses and deck thickness vs girder spacing used by other DOTs in the U.S. This questionnaire will be mailed to all 50 states, and the resulting information presented for comparative purposes.

4. Meet with 2 bridge contractors in the state to identify the primary effects of increasing deck thickness on construction issues and cost of the deck and other bridge components. Also, the relative merits/demerits of ALDOT employing a standard deck thickness of 8" will be discussed form construction and cost viewpoints. Uniformity of deck thickness, e.g. 8", would allow standardization of rebar mats or girds support chairs in attaining

proper bottom and top covers, would simplify construction and inspection, and should result in enhanced deck quality/durability.

5. Perform a parameter sensitivity study to assess the effect of deck thickness on design and construction parameters. Parameter tables and graphs will be prepared to present the results of the sensitivity study.

6. Analyze and synthesize the results of 1-5 above to assess the primary effects of increasing deck thickness, and to identify optimum value(s) of deck thickness based on cost/benefits. Draw appropriate conclusions and make recommendations on bridge deck thickness (es) to maximize cost/benefits.

2. BACKGROUND AND LITERATURE REVIEW

2.1 Background

Bridge decks must withstand one of the most damaging types of live load forces, i.e., the concentrated and direct pounding of truck wheels. One of the functions of the deck is to distribute these forces in a favorable manner to the support elements below. The ratio of live to total load stresses is high in bridge decks, usually much higher than in most of the other components of the bridge, and such fatigue-producing stresses tend to aggravate any defects that might be present in the deck (10). Additionally, because of its exposed location, temperature variations are large in bridge decks and restraints to the resulting volume changes tend to cause early cracking of the concrete as well as fatigue-producing stresses.

Many bridge decks constructed in the U.S. in the 1950's and 1960's suffered from severe cracking and deterioration. These decks were typically in the 6" - 6.5" thickness range. In the 1970's many state DOTs enacted deck design changes to address these premature deterioration and poor durability performances. The primary changes were increased deck thickness (to approximately 8" - 8.5") and increased cover on the deck top reinforcing steel to 2" - 2.5"(30). Texas, New York and New Jersey were three of the states making such changes and in talking with their bridge engineers; most were pleased with the improved deck performances. It is interesting to note that the ACI recommends a nominal deck thickness of 8" (1).

Alabama has many bridges, which have good substructures and superstructures, but with badly cracked and deteriorating decks as can be seen in Figs. 2.1 and 2.2. It appears that these cracks are primarily the result of (24)

- early drying and thermal shrinkage
- early concrete obstructed settlement

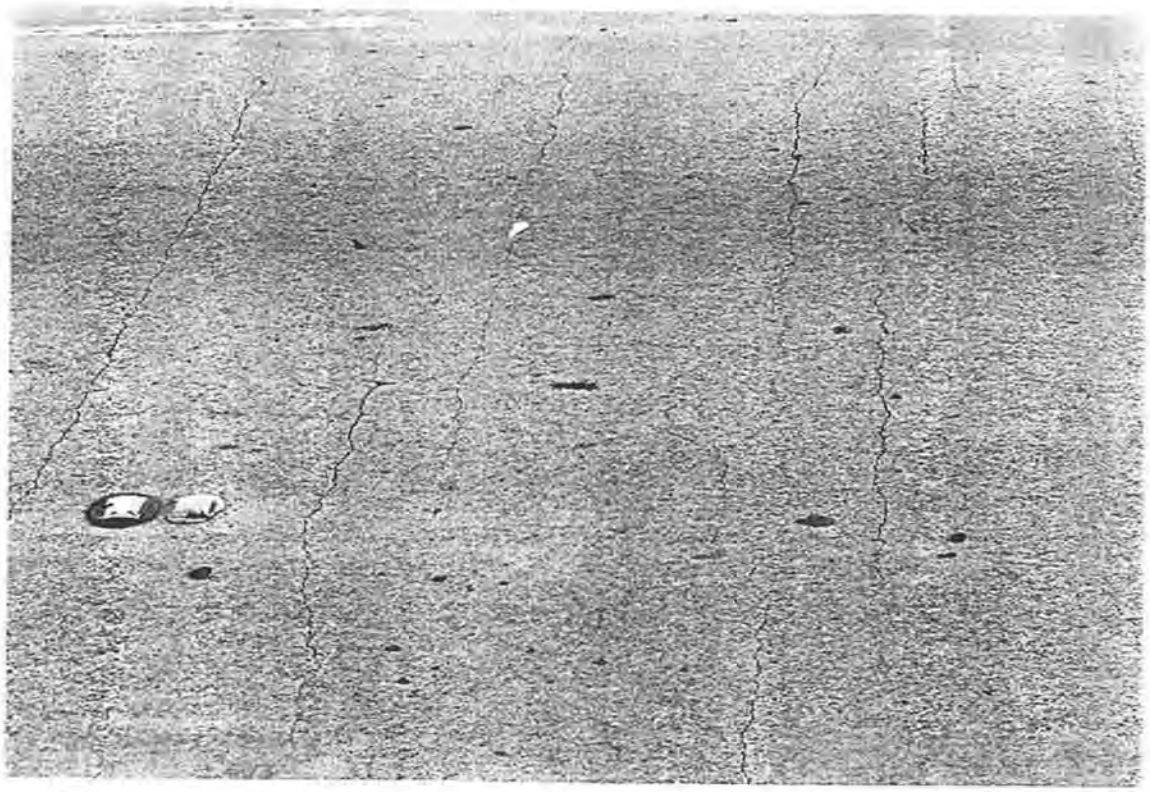


Fig. 2.1 Close-up of Transverse Cracking on I-65 Bridge in Birmingham, AL (22)

Fig. 2.2 I-65 Deck Surface Spall in Birmingham, AL (22)

- thin and flexible deck (approximately 6½" thickness)
- light and flexible superstructure
- heavy traffic volume and truck loading

The typical top-down failure chronology for bridge decks in Alabama appears to be as follows (24):

- A significant level of early transverse shrinkage cracking
- Growth in width of transverse cracks due to crack movement and abrasion from traffic and environment loadings
- Development of longitudinal cracks at girder edges due to poor longitudinal distribution of truck tire loadings (due in part to extensive transverse cracking)
- Reduced bending stiffness in both the transverse and longitudinal directions due to crack growth which in turn leads to increased deck cracking
- Small regions of the deck top surface become "boxed-in" by transverse and longitudinal cracks, and local top surface delaminations and spalling occur from the pounding traffic
- Local surface spalling causes greater traffic impact loadings which lead to further delaminations and spalling and require ever increasing maintenance attendance
- Eventual deck punching shear failures

2.2 Literature Review

Primary Findings of UAB Study. Researchers at the University of Alabama at Birmingham (UAB) (9) performed condition surveys as well as analytical and experimental analyses on 10 Birmingham bridge decks (5 with severe cracking damage and 5 largely undamaged for control) during the period 1993-1995 to determine the cause of the significant deck deterioration. General information about the bridges investigated is provided in Table 2.1. As can be seen in the fourth column of Table 2.1, the average age of the bridges studied was approximately 40 years at the time of the study.

Table 2.1 General Information About UAB Decks Studied (22)

Damage Category	Study No.	Location	Const. Date (age)*	Span Type/ Length(s)	Girder Type	Deck Cond. Rating	Deck Damage	Transverse Crack Spacing (Crack Width)
Severely Damaged Decks	1	I-59 over Little Canoe Creek	1961 (34)	simple 34'	RC	3	advanced deck damage, sudden localized deck failures	24" (0.04")
	2	I-65 over US 31 near Blountsville	1959 (36)	continuous 30 degree skew 60'/55'/59'	RC	4	severe transverse cracking	24" -
	3	US 78 over Locust Fork of the Warrior River	1962 (33)	continuous 141'/180'/140'	steel	5	severe transverse cracking	18" - 30" (0.05")
	4	AL 269 over TCI Railroad	1954 (41)	continuous 134'/164'/225'	steel	5	moderate transverse cracking	55" (0.04")
	5	AL 269 over Copeland Ferry	1955 (40)	continuous 184'/184'	steel	5	severe transverse cracking	26" (0.05")
Undamaged Decks	6	AL 79 over Five Mile Creek	1956 (39)	simple 45 degree skew 34'	RC	6	minimal damage (spalling @ exp. jt.s)	-
	7	US 78 over Kelly Creek	1955 (40)	simple 45 degree skew 46'	RC	8	minimal damage	-
	8	US 31 over Bishop Creek	1955 (40)	simple 42'	RC	7	minimal damage	-
	9	US 31 over Black Creek	1955 (40)	simple 40'	RC	7	minimal damage	-
	10	AL 269 over Southern Railroad	1945 (50)	simple 36'	RC	7	minimal damage	-

*At time of survey in 1995.

Table 2.2 Summary of UAB Condition Survey Results (22)

Damage Category	Bridge Study No.	Measured vs. Required Deck Thickness ($D_{meas.}/D_{req.}$)	Measured Deck Span (Transverse) to Thickness Ratio ($S/D_{meas.}$)	Measured vs. Required Concrete Strength ($f'_{c\,meas.}/f'_{c\,req.}$)	Design Tensile Stress vs. Modulus of Rupture ($f_{t\,Design}/f'_t$)	Total Truck Loading Intensity ($\Sigma(W_{recorded}/W_{legal})$)	Measured vs. Required Distribution Rebar Spacing ($S_{meas.}/S_{req.}$)	Measured vs. Required Shrinkage/ Temperature Rebar Spacing ($S_{meas.}/S_{req.}$)	Concrete Coarse Aggregate
Severely Damaged Decks	1	0.81	14.9	1.25	2.22	5,500	1.2	0.9	smooth quartz and sandstone gravel
	2	0.85	14.5	-	1.81	10,500	1.9	-	polished quartz gravel
	3	1.00	10.7	1.75	1.24	20,500	1.5	1.5	smooth sandstone gravel
	4	0.81	20.6	1.70	2.07	2,500	2.4	1.8	slag
	5	0.88	13.4	1.88	1.59	2,500	1.7	1.2	smooth sandstone gravel
Undamaged Decks	6	0.80	13.0	2.15	1.61	7,500	1.2	0.9	crushed limestone
	7	0.96	10.9	-	1.11	100	1.6	0.8	pea gravel
	8	0.96	10.9	2.73	1.00	1,200	0.9	1.0	crushed limestone
	9	0.88	11.8	2.10	1.35	800	1.2	0.7	slag
	10	0.88	11.8	2.14	1.34	500	1.2	0.7	slag

Some of the primary results of the UAB condition surveys are summarized in Table 2.2. As can be seen in Table 2.2, the bridges with the severely damaged decks were on the unfavorable side relative to the required values for most of the conditions shown. Relative to the undamaged decks, the bridges with the damaged decks were on the unfavorable side of every condition assessed.

The UAB researchers determined that severe transverse cracking at the top surface above the transverse rebars was the predominant form of deck damage. Their assessment of the primary causes of the severe transverse cracking, and their recommended corrective actions were:

- Excessive deck slenderness, i.e., excessively large values of deck span/thickness ratios. The design thickness for 7 of the 10 decks was 6 1/4", and was 6 1/2" for 2 others. Even worse, the mean as-constructed thickness for these 9 decks was 5.61", with a range of 5" - 6 1/4". So, indeed the decks are very thin. The researchers recommended increasing the deck thickness so that the design tensile stresses do not exceed the concrete cracking strength.

- Excessive truck loadings in number of trucks and in weight of trucks.

- Insufficient longitudinal rebar in both the top layer (shrinkage and temperature rebar) and the bottom layer (load distribution rebar). The researchers recommended increasing the percentage of top longitudinal rebar and in placing it on top of the transverse steel to improve its effectiveness and to reduce transverse cracking over the top transverse bars.

- Poor quality, gradation, and shape of the deck concrete coarse aggregate. Decks of hard, crushed and well graded aggregate performed better than decks of softer/absorptive, rounded (river gravel) and poorly graded aggregate. The researchers found that river gravel (rounded aggregate) was used as the coarse aggregate in 4 of the 5 severely damaged decks. They recommended using only crushed stone coarse aggregate, and that coarse and fine aggregate be well graded.

- Poor construction QC/QA by construction contractors and ALDOT inspectors. Large variations in deck thicknesses, rebar spacing and rebar cover were detected during the field condition surveys. In most every case, the variation or error was on the side of reducing the deck strength, stiffness and durability. The researchers recommended additional or improved inspection be conducted during construction to assure the as-built bridge is the same as shown in the construction documents.

Some significant results of the UAB study which are germane to this study are:

- The Birmingham bridge decks are quite thin and are badly cracked. This would imply that thicker decks are probably in order.

- The deck construction and inspection quality leave much to be desired. Note in Column 3 of Table 2.2 that 9 out of 10 of the decks are thinner than specified with 3 of these having deck thicknesses of around.

$$D_{actual} = 0.80\ D_{specified}$$

$$= 0.80 \times 6.25'' = 5.00''$$

  A 1 1/4" error in thickness on a deck specified to be 6 1/4" thick is an unacceptable construction tolerance. Also note in Columns 8 and 9 of Table 2.2 that the errors in horizontal location of the deck longitudinal reinforcing steel (distribution steel in the bottom and shrinkage/temperature steel in the top) were gross and of unacceptable construction tolerance for most of the bridges. It can only be assumed that if the horizontal location of the reinforcing steel is so poor, then the vertical location and concrete cover on the steel, are probably equally as poor. The UAB researchers did not show tables of values of deck rebar cover measured in the field, however they did indicate numerous instances of smaller that specified rebar cover.

  Though this is a construction QA/QC problem, from a practical standpoint, designing the decks somewhat thicker would render them more "forgiving" of poor construction and inspection quality. That is, it would be probably be wise to employ a "belt-and-suspenders" approach in avoiding situations such as this in the future. Also because of the large deviation between the specified and as-built deck thickness and rebar locations, the ALDOT should closely examine its procedures for estimating deck deflections during placement of the deck concrete, vertical displacements of the deck rebar mats relative to the deck, and for achieving the proper cover on the top and bottom rebar mats.

- On the positive side, Columns 5 and 6 of Table 2.2 indicate that the in-place concrete was superior (based on strength) to that specified.

Bridge Deck Cracking. As evident in the typical bridge deck failure chronology given earlier, bridge deck cracking is the primary cause of deck deterioration and reduced longevity in Alabama. The dominant type of bridge deck cracking is transverse cracking in the top of the deck above the top transverse reinforcing steel. This type of cracking is predominately caused by

(1) early thermal and drying shrinkage cracking, and

(2) obstructed settlement of the deck concrete as illustrated in Figures 2.3 and 2.4.

Later it will be shown that deck thickness does not significantly affect deck thermal and drying shrinkage cracking. However, for a given cover requirement on the deck reinforcing steel, thin decks will tend to develop obstructed settlement cracks more readily due to the concrete not being able to be consolidated as effectively because of the closeness of the two reinforcing steel mats.

After initial placement, vibration, and finishing, concrete has a tendency to continue to consolidate. During this period, the plastic concrete may be locally restrained by reinforcing steel, a prior concrete placement, or formwork. This local restraint may result in voids and/or cracks adjacent to the restraining element as indicated in Figure 2.3. It should be noted that where the top and bottom mat rebar are vertically aligned over the small thickness of a bridge deck, any significant obstructed settlements and cracks will create a significant vertical plane of weakness as illustrated in Figure 2.4.

The primary design parameters affecting cracking and creation of planes of weakness during obstructed settlement are

- concrete cover-cracking decreases sharply with increased cover especially in 1" to 2" cover range.
- concrete slump-cracking increases with slump.
- rebar size-cracking increases with rebar size.
- rebar splices-rebar splices are equivalent to using a larger diameter rebar and thus increase cracking.
- construction parameters such as insufficient vibration, leaking forms, or highly flexible forms increase obstructed settlement cracking.

Dakhil et al. (12) identified and quantified the primary design parameters affecting settlement cracking. Results from their work are shown in Table 2.3 and Figure 2.5. Note that ALDOT's

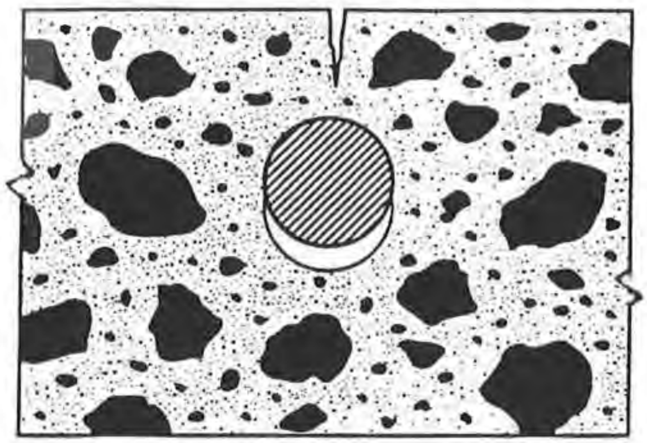


Figure 2.3. Crack Formed due to Obstructed Settlement (Price 1982) (3)

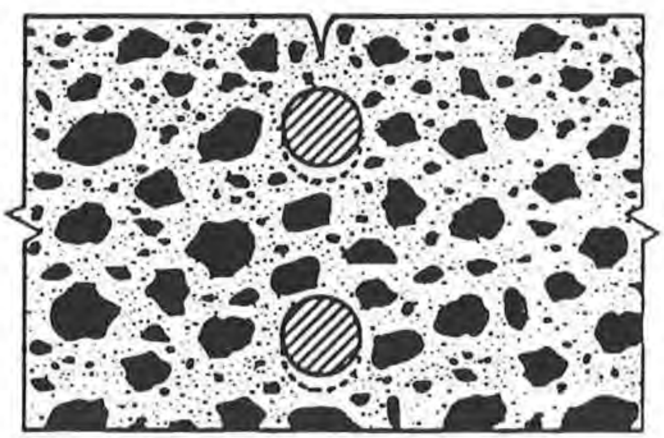


Figure 2.4. Vertical Plane of Weakness due to Obstructed Settlement

Table 2.3 Probability of Subsidence Cracking Predicted by Regression Analysis (12)

	Probability of Cracking[1] (percent)								
Slump (in.)	2			3			4		
Bar Size / Cover (in.)	#4	#5	#6	#4	#5	#6	#4	#5	#6
¾	80.4	87.8	92.5	91.9	98.7	100.0	100.0	100.0	100.0
1	60.0	71.0	78.1	73.0	83.4	89.9	85.2	94.7	100.0
1½	18.6	34.5	45.6	31.1	47.7	58.9	44.2	61.1	72.0
2	0.0	1.8	14.1	4.9	12.7	26.3	5.1	24.7	39.0

[1]Computed probability values of less than 0 percent or greater than 100 percent are reported as 0 percent or 100 percent, respectively.

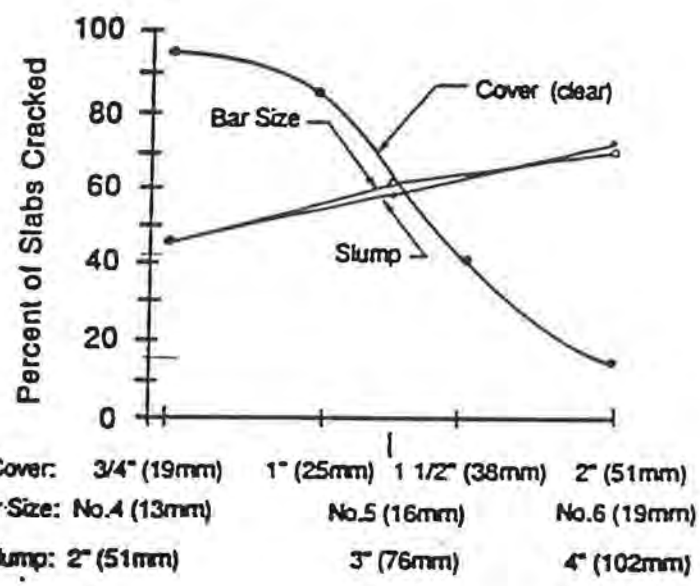


Figure 2.5. Settlement Cracking as a Function of Bar Size, Slump and Cover (12)

practice of using 2 in of cover, small diameter rebar, and low slump concrete is very good for mitigating deck settlement cracking. However, note in Figure 2.5 the great sensitivity of slabs to settlement cracking for concrete cover less than 2 in, e.g., if the cover is 1.5 in. rather than 2.0 in., the percentage of slabs cracked will increase from around 18% to 42%. This coupled with the great variation in as-built rebar location (due to contractor error and due to overestimating allowances for girder deflections when placing the slab) as documented by the UAB study (9), it would probably be wise to increase the cover on the top reinforcing steel in decks from 2 in to 2.5 in. Doing this, a construction error of -0.5 in. in the concrete cover would still yield a slab with low settlement cracking.

It can be noted that the Penn DOT concluded from a early 1970's study that horizontal fracture planes and spalls, transverse cracking, and surface mortar deterioration were the principle forms of deck deterioration. Increasing the cover on the top mat from 2 in to 2.5 in should be helpful in mitigating horizontal fracture planes and spalling of the deck top. Also, Canadian researchers and transportation agencies (11,14) and the Colorado DOT (8) have recently shown that loads on bridge decks are transmitted to the supporting girders, primarily via arch action of the slab rather than flexural beam action. In this case, increasing the deck thickness would improve the decks arch action capabilities (as well as its flexural capabilities) and in turn would result in lower deck stresses and greater fatigue life. Also, increasing deck thickness, and in the process increasing the top rebar cover from 2 in. to 2.5 in., would provide greater corrosion protection of the top steel as deicing salts are used with greater frequency, and it would allow the decks to be ground 0.5 in. during their service life to enhance rideability and/or skid resistance.

Also, 1/6.6 - scale laboratory load tests by Perdikaris (21) found an extensive gridlike cracking pattern on the bottom side of their model decks when using a moving wheel-load. Their cracking pattern was similar to that observed on the bottom side of the Birmingham bridge decks. Perdikaris also reported that load tests performed by Ockleston in 1956 showed slab collapse loads, which were 3-4 times the capacities predicted by "yield-line" theory. The enhanced slab ultimate strengths were attributed to "dome" or "arching" action of compressive membrane forces present in slabs when the supports are restrained against lateral movement.

Perdikaris et.al (22) concluded, based on a series of 1/3 and 1/6.6 model tests and resulting cracking load level and steel strain measurements, that truck wheel loads in the U.S. are not likely to affect the serviceability and integrity of reinforced concrete bridge decks.

The strain induced in a material (when unconstrained) by a change in temperature is,

$$\varepsilon_t = \alpha \, \Delta T \tag{2.1}$$

where α = coefficient of thermal expansion

ΔT = change in temperature from datum value, i.e., from temperature of concrete at time of setting.

If the material is unconstrained, the associated thermal stress is zero.

If the material/member is fully constrained, then the thermal strain is zero, but the associate thermal stress assuming elastic behavior is

$$\sigma_t = -E\alpha \, \Delta T \tag{2.2}$$

where E = Young's Modulus.

Figure 2.6 illustrates the effects of thermal 'loadings' on movements, strains and stresses in a bridge component for a uniform and linear varying temperature change.

An exposed concrete bridge deck is heated or cooled by the following (Radolli & Green):

1. Convection to or from the surrounding atmosphere,
2. Radiation from the sun, and
3. Radiation to or from the sky or surrounding objects,

as shown in Figure 2.7.

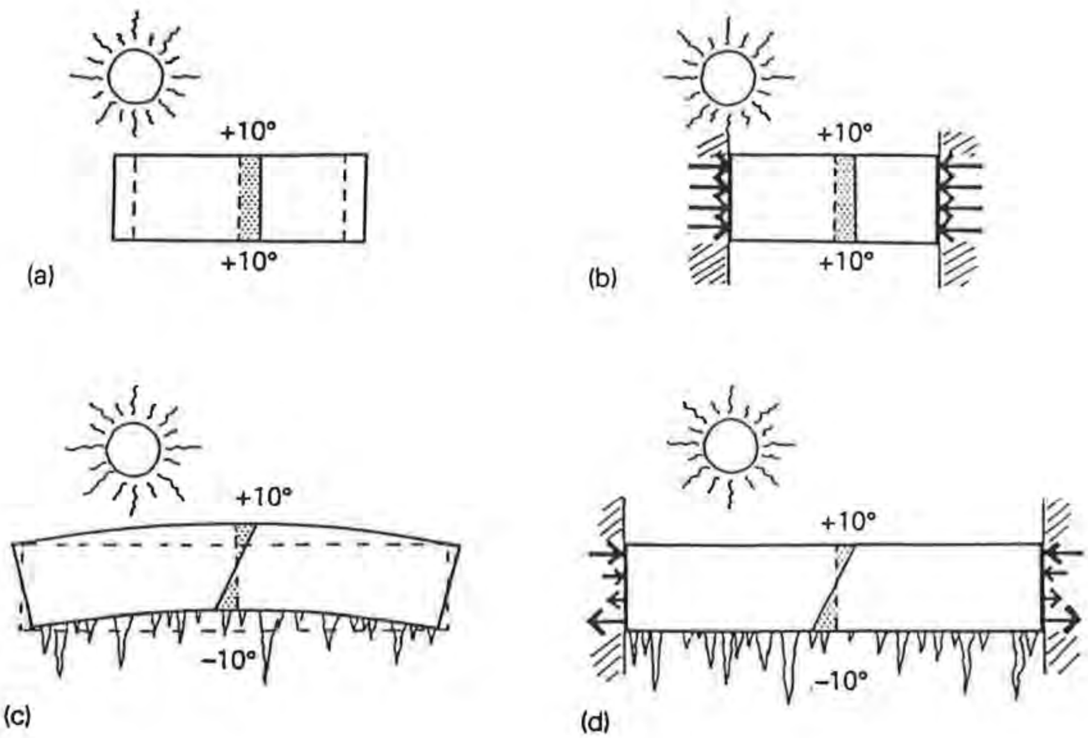


Fig. 2.6 Effects of Temperature Distribution and Restraint Conditions on Movements and Stresses in Elements of Bridge Deck (16)

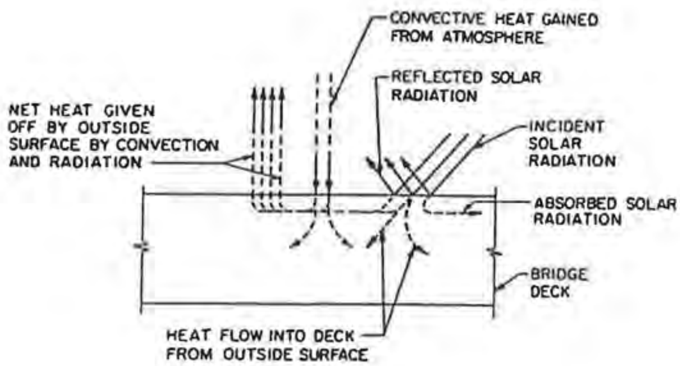


Figure 2.7. Heat flow in exposed bridge deck (Ref. Unk.).

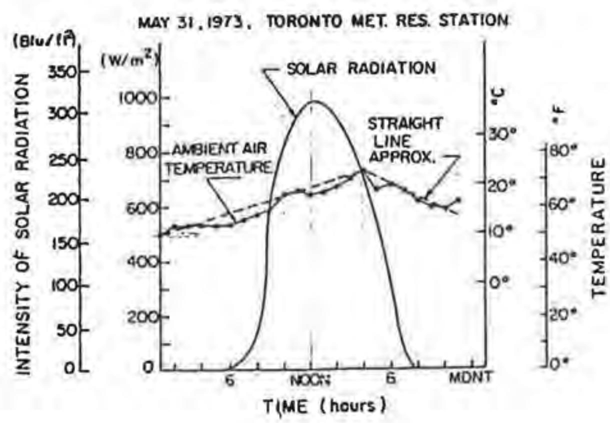


Figure 2.8. Intensity of solar radiation and range in ambient air temperature (Ref. Unk.).

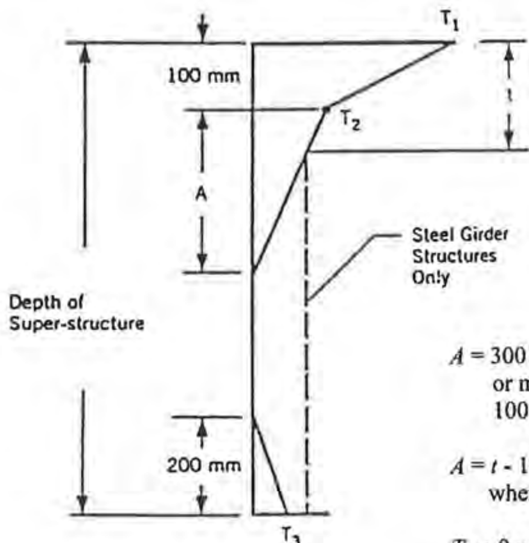
Maximum intensity of solar radiation occurs in the summer and generally increases with increasing proximity to the equator. Available data indicates a maximum intensity of incoming solar radiation on a horizontal surface occurs between 11:00 a.m. and 2:00 p.m.

Radiant energy from the sun is partially reflected and partially absorbed. Absorbed energy heats the bridge deck surface and gives rise to a temperature gradient through the deck. Some of the absorbed radiant energy is lost from the surface by convection and reradiation as indicated in Figure 2.7. Values of solar radiation and ambient air temperature observed in Toronto, Ontario, on May 31, 1973 are shown in Figure 2.8. Values of average solar radiation for the May to July period are similar for the United States and southern Canada, and hence the data shown in Figure 2.8 are representative.

Deck positive thermal gradients (deck top temperatures are larger) are a function of solar radiation gain at the deck top surface. AASHTO specifications partition the U.S. into solar radiation zones as shown in Fig. 2.9. Design thermal gradients are determined using the solar radiation zones indicated in Fig. 2.9 and then using Fig. 2.10. It should be noted that increases in deck temperature and positive thermal gradient typically do not cause deck tensile stresses and cracking. They can cause increased deck concrete temperature at set-time, and in turn this will increase thermal tensile stresses when deck temperatures fall (both ΔT_{AVG} and $\Delta T_{gradient}$ (negative gradient)). Thus, it is the decreases in deck temperature from that at set-time, which cause tensile stresses to form in the concrete and create the potential for cracking. This is true of uniform decreases in the deck temperature (relative to the support girders), and negative thermal gradients (deck top temperatures are lower than underside).

When working with concrete, drying shrinkage strains and stresses can be determined in the manner described above for thermal strains and stresses, where the drying shrinkage strains can be

Figure 2.9. AASHTO Solar Radiation Zones (6)



A = 300 mm for closed concrete structures that are 400 mm or more in depth. For shallower sections, A shall be 100 mm less that the actual depth.

$A = t$ - 100 mm for steel superstructures, where t = thickness of the concrete deck.

T_3 = 0 unless site study indicates otherwise, but is not to exceed 3° C.

Gradient Temperatures [a]

	Concrete Surface		50-mm Asphalt		100-mm Asphalt	
Zone	T_1	T_2	T_1	T_2	T_1	T_2
1	30	7.8	24	7.8	17	5
2	25	6.7	20	6.7	14	5.5
3	23	6	18	6	13	6
4	21	5	16	5	12	6

[a]In degrees celcius (°C).

Figure 2.10. AASHTO Design Temperature Gradients (Modified from Ref. 6)

treated as an equivalent temperature change, i.e.,

$$\Delta T_{eq} = -\frac{\varepsilon_{sh}}{\alpha} \tag{2.3}$$

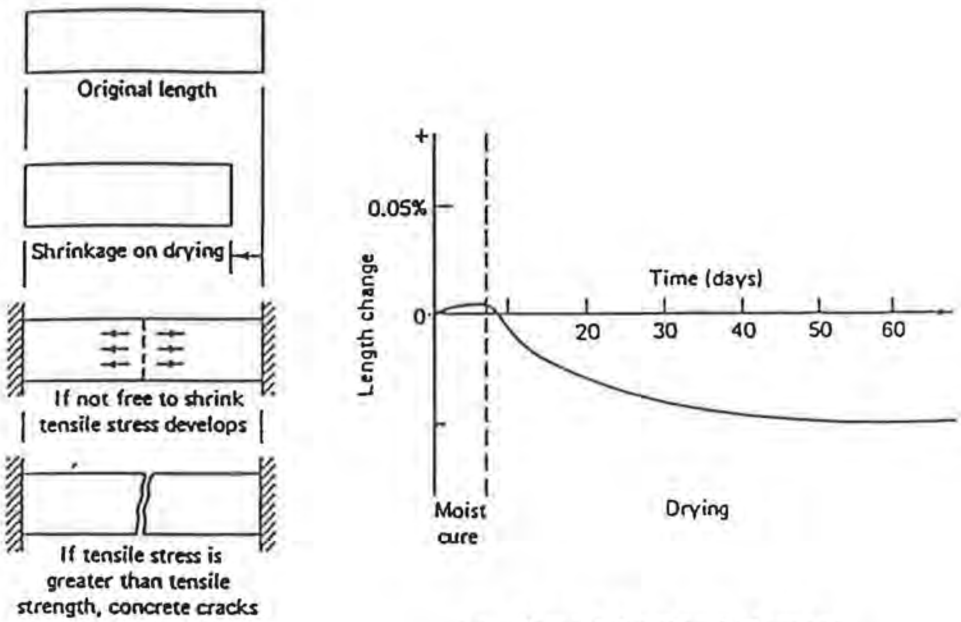
where ε_{sh} = drying shrinkage strain.

Contractions (shrinkages) are considered positive, but would require a decrease in temperature, thus the necessity for the negative sign in Equation 2.3. Figure 2.11 shows the behavior of a bridge component under drying shrinkage for two materials with different drying shrinkage characteristics.

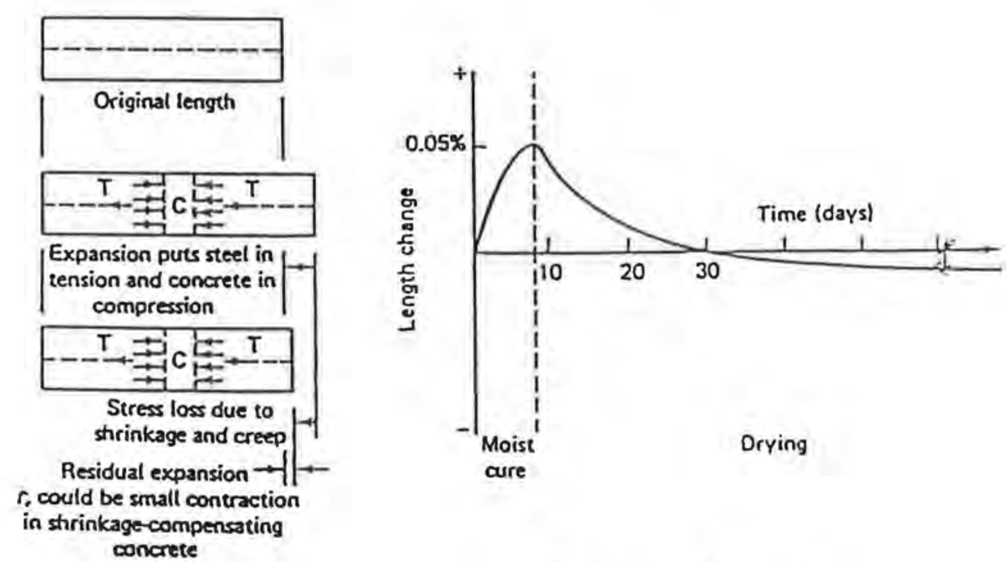
In the transverse direction of a bridge deck, in the worst case scenario (for stresses and possible decking cracking), for simple thermal 'loadings' such as shown in Figure 2.6, the thermal stress would be approximately as shown in Figure 2.6. That is, due to symmetry and the transverse diaphragms, thermal stresses would be approximately as indicated in Figure 2.12. Note that the thermal stresses shown in Figure 2.12 are independent of the deck thickness and only depend on the thermal 'loading', i.e., the ΔT distribution (which is primarily independent of the deck thickness), and the E and α of the deck concrete.

Wolff (34) used stress equations derived by Zuk (37) and a second degree equation for temperature and drying shrinkage variation through the depth of deck, and evaluated the thermal and shrinkage stresses shown in Table 2.4. These stresses were for the temperature and drying shrinkage distributions shown in the table, and for a girder spacing of 8 ft. Obviously Wolff's work indicates that deck thermal and drying shrinkage stresses are effectively independent of deck thickness.

Willson (33) used a more sophisticated temperature distribution and larger temperature variations through the depth of a simply supported bridge deck - superstructure system consisting of Bulb - T72 girders at a spacing of 6'-8½". The results of his numerical analyses pertaining to the effect of deck thicknesses on longitudinal stresses are shown in Fig. 2.13. For the positive



(a) Type I Portland Cement



(b) Expansive Cement

Fig. 2.11 Drying Shrinkage Behaviors of Bridge Component (20)

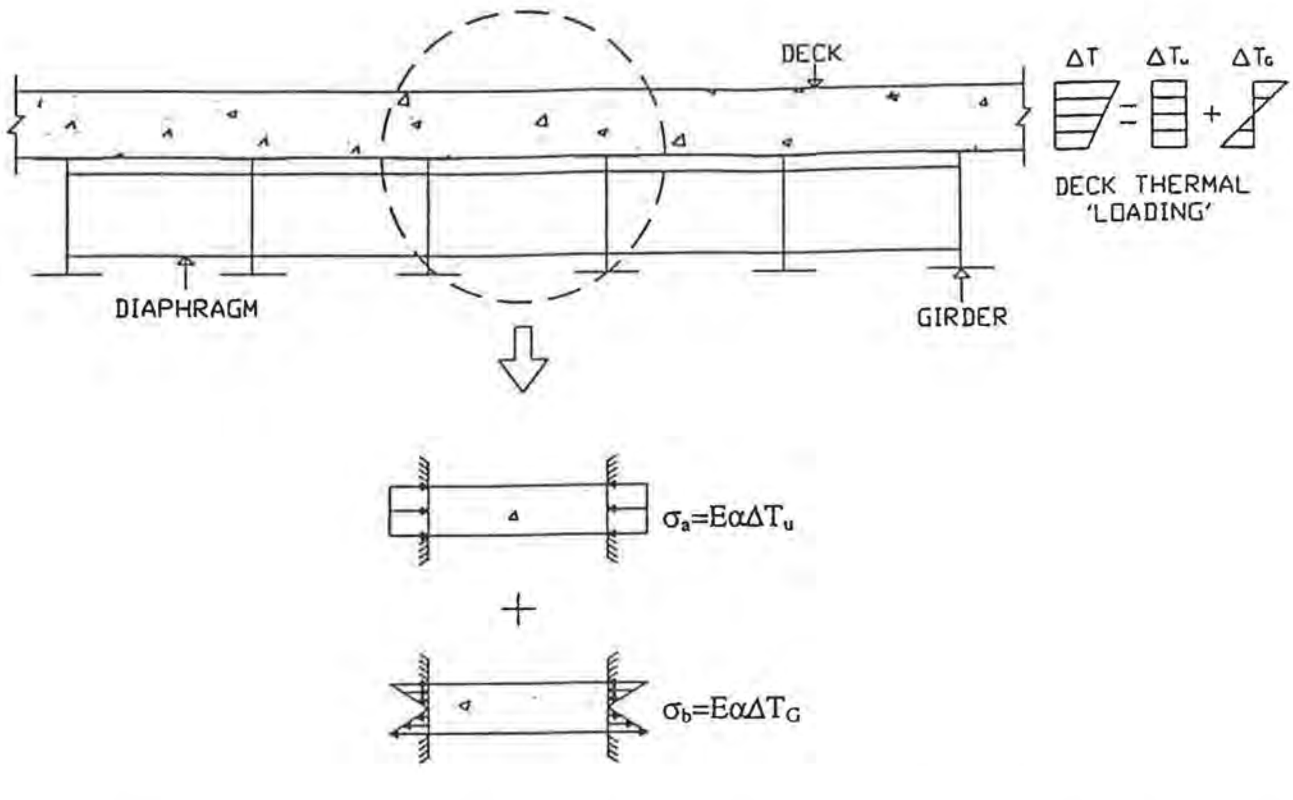
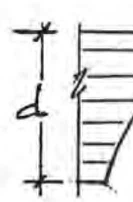


Fig. 2.12 Worst Case Deck Stresses in Transverse Direction Due to Simple Thermal Loading

Table 2.4. Effect of Deck Thickness on Deck Thermal and Shrinkage Stresses (modified from (34))

'Loading'	Deck Thickness, d (in)	Max Stress $\sigma_{long.}$ (psi)	$\sigma_{trans.}$ (psi)
 $\Delta T_T = +5°F$, $\Delta\varepsilon_{Sh,T} = 600x10^{-6}$; $\Delta T_B = -10°F$, $\Delta\varepsilon_{Sh,B} = 0$	6	686	858
	7	671	855
	8	659	852
	9	649	850

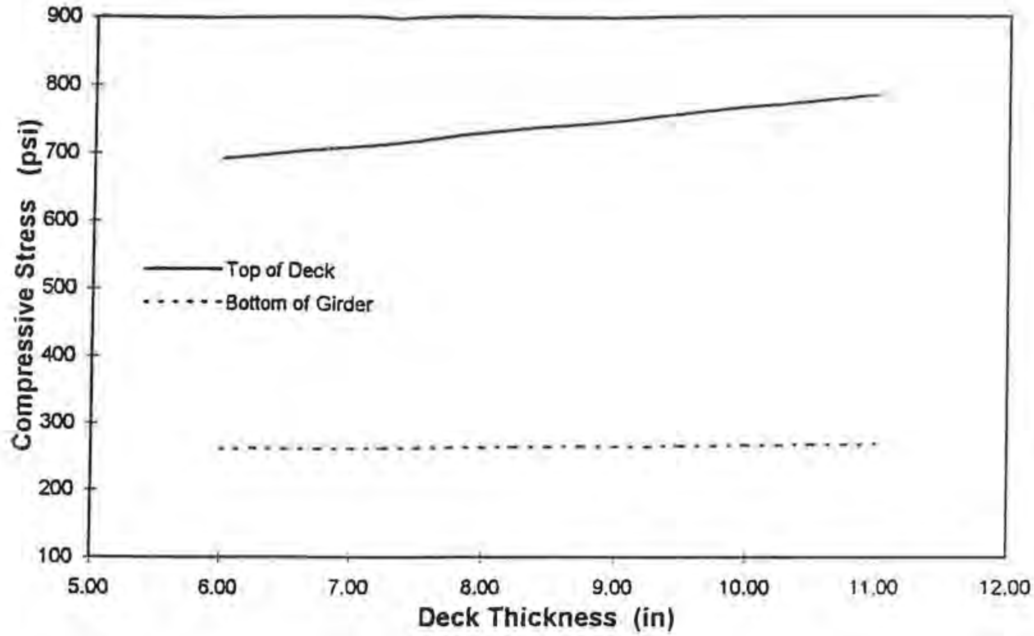
temperature gradient (temperature at top of deck is higher than that at the bottom) shown in Fig. 2.13a, the bottom of the girder stresses remain essentially constant and independent of deck thickness. The top of deck longitudinal stresses increase from 690 psi to 784 psi as the deck thickness is increased from 6" to 11". This is a 19 psi increase in stress per inch of increase in deck thickness which is negligible. For the negative temperature gradient (temperature at top of deck is lower than that at the bottom) shown in Fig. 2.13b, the deck thermal stresses increase slightly with deck thickness, but the combined thermal plus shrinkage stresses decrease slightly (8 psi per inch of deck thickness) with deck thickness.

Thus for more complex thermal 'loading' and boundary conditions, the analyses are more difficult; however the stresses remain primarily independent of the deck thickness. In the longitudinal direction, one must bring in the properties and contributions of the longitudinal girders. However, again in this case the deck thermal stresses are primarily independent of the deck thickness.

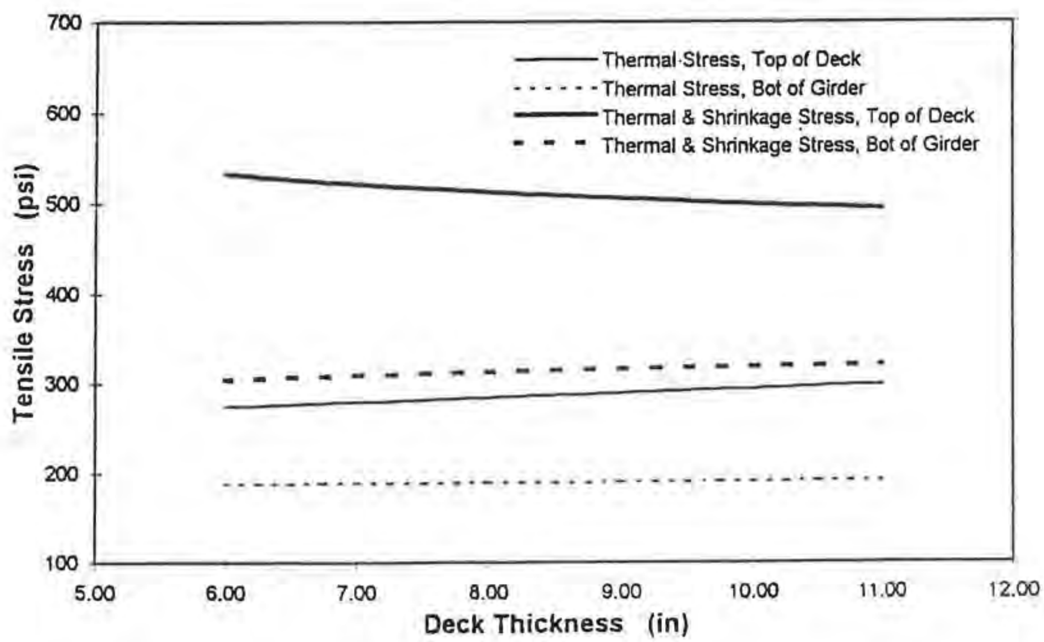
Since drying shrinkage behavior can be analyzed as a thermal behavior under an equivalent thermal 'loading' as indicated in Equation 2.3, then deck shrinkage stresses will likewise be primarily independent of the deck thickness. However, the drying shrinkage strain, ε_{sh}, in Equation 2.3 is not a constant, but varies with a number of parameters.

Mindess and Young (20) state "that the size and shape of a concrete specimen will determine the rate and amount of drying shrinkage." They report the effects of volume-to-surface ratio on the drying shrinkage of concrete to be as shown in Fig. 2.14. Note that in case of bridge decks,

$$Volume - to - Surface - Ratio = \frac{S_{area} \cdot t}{S_{area}} \approx t \qquad (2.4)$$



a. Compressive Stress For Positive Temperature Gradient



b. Tensile Stress For Negative Temperature Gradient

Fig. 2.13 Longitudinal Stress Versus Deck Thickness for Simply Supported Bridge (33)

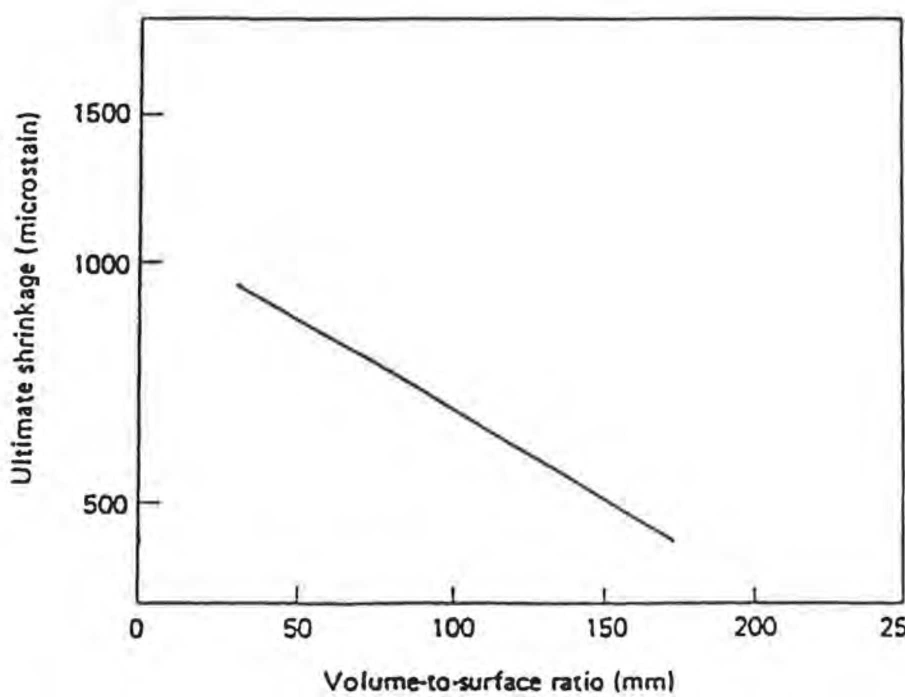


Fig. 2.14 Variations of Drying Shrinkage with Concrete Volume-To-Surface Ratio (20)

Thus, if a 7" (178 mm) deck thickness is increased to 8" (203 mm), Fig. 2.14 indicates there would be a significant reduction in the ultimate drying shrinkage strain.

Longitudinal cracks are usually caused by truck loadings causing transverse flexure of thin decks with wide girder spacing. These decks have a greater flexibility than thicker ones, and the flexure cracks occur when the cracking moment capacity of the slab is exceeded. The cracking moment capacity can be calculated by,

$$M_{cr} = 7.5\frac{I_g}{y^t} \tag{2.5}$$

where I_g is the gross moment of inertia of the slab and y^t is the distance from the centrodial axis of the gross section to the extreme fiber in tension.

To increase the cracking moment capacity either the concrete tensile strength has to be increased or the deck section modulus (I_g/y^t) has to be increased. Since the deck section modulus varies as the square of the thickness, increasing the deck thickness is an excellent means of increasing its cracking moment capacity and mitigating longitudinal deck cracking.

Krauss and Rogalla (19) report on a recent comprehensive investigation of transverse cracking in newly constructed bridge decks, and conclude, based on its literature review, that "decks should not be less than 8 to 9 in. thick."

Typically, transverse deck cracking leads to more transverse cracking, which tends to lead to longitudinal cracking. When small regions of the deck top surface become 'boxed-in' by transverse and longitudinal cracks, top surface delaminating and spalling occur from the pounding traffic (see Fig. 2.2). In turn, the resulting "potholes" cause greater traffic impact loading which lead to further delamination and spalling. Thus is the typical top down-deck deterioration process in Alabama.

Surface spalling is one of the dominant bridge deck durability and maintenance problems today. Stark (29) used pachometers and studied 7 bridge decks in Michigan to assess the relationship of deck cracking, rebar cover, and surface spalling. He summarized the primary findings of his study as follows:

(1) All spalls were associated with reinforcing steel with less than 2" of cover (usually the cover was less than 1½"). Numerous areas with 2" or more of cover were found adjacent to the areas with 1½" or less of cover, and no spalls were found in these areas of greater concrete cover.

(2) Petrographic examinations of concrete cores from other states (Kansas, Missouri and California) also revealed that spalls and incipient spalls were associated only with steel having less than 2" of cover.

(3) Spalls occurred only in areas where vertical cracks occurred directly over the top reinforcing steel. In areas where only pattern or random cracks, or no cracks, occurred, there was a total absence of spalls.

(4) Where the cover was less than 2", the crack pattern consisted primarily of cracks directly above the top steel, but where there was more than 2" of cover, random or pattern cracking was most often found.

Stark's work would indicate that going to a 2 ½" cover (to allow for construction tolerances), or to place the top longitudinal bars on the top, and thus achieve a 2 ½" cover on the top transverse bars, maybe a wise course of action to mitigate deck spalling and maintenance problems.

Punching Shear Failure. Csagoly (11) recently reported the results of a laboratory testing program to assess the behavior and failure mode of concrete bridge decks. They used reduced scale test models and load tested with a concentrated load (to simulate a truck tire loading). They found that as the deck deflected, the internal deck arch became shallower and increased the deck concrete arching stresses. When the deflection was half of the deck depth, the arch essentially flattened out and became compressively unstable under further loading. When the maximum deflection exceeded half the deck depth, punching shear failure occurred (see Figs. 2.15 - 2.17).

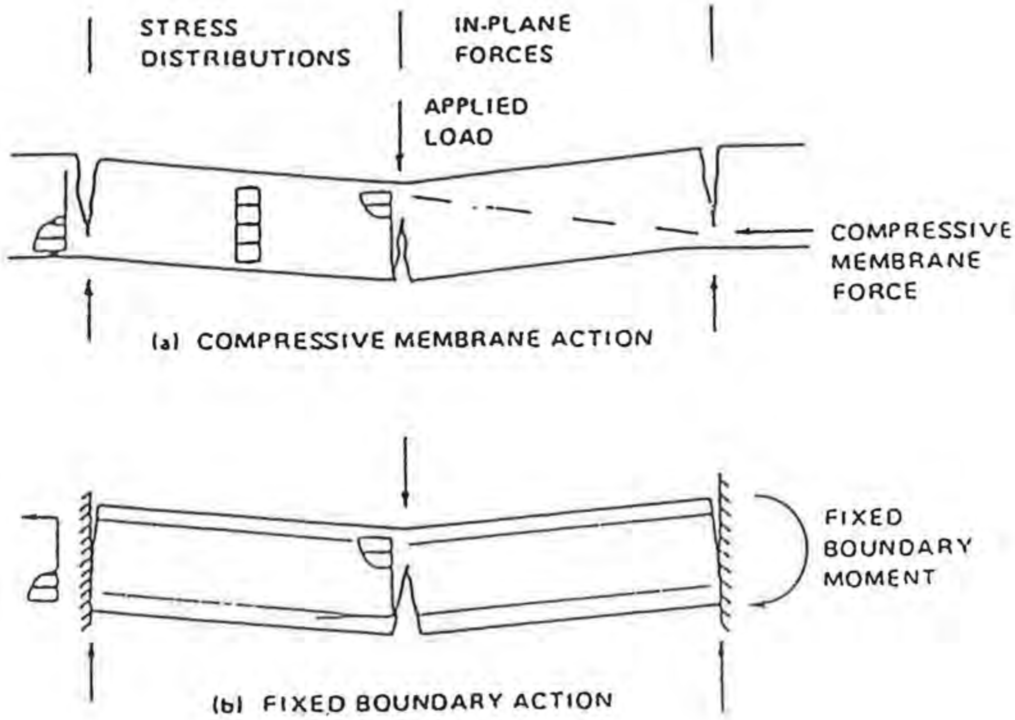


Figure 2.15 Arching Action in Deck Slabs (11)

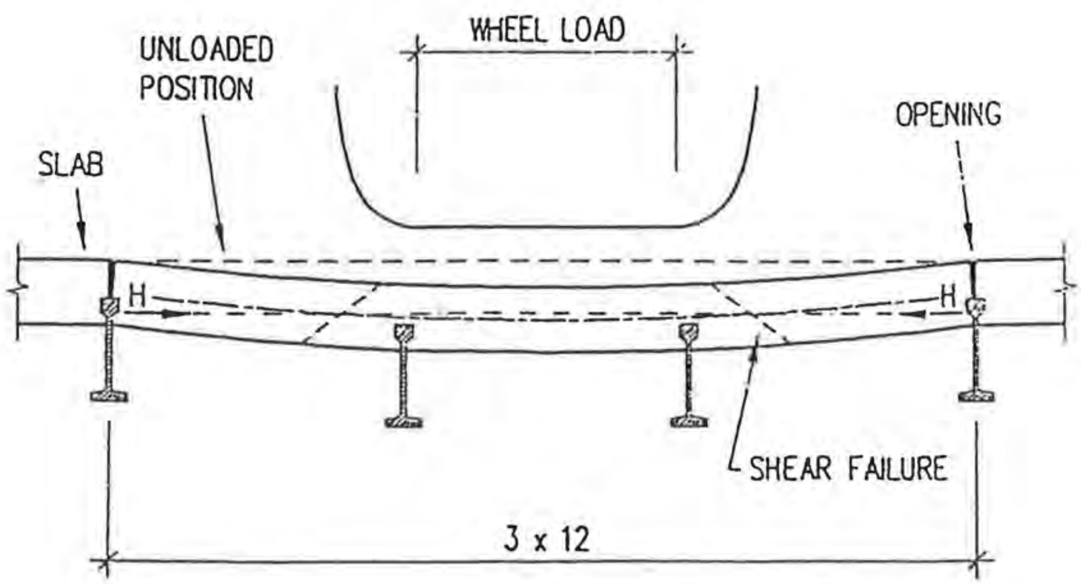
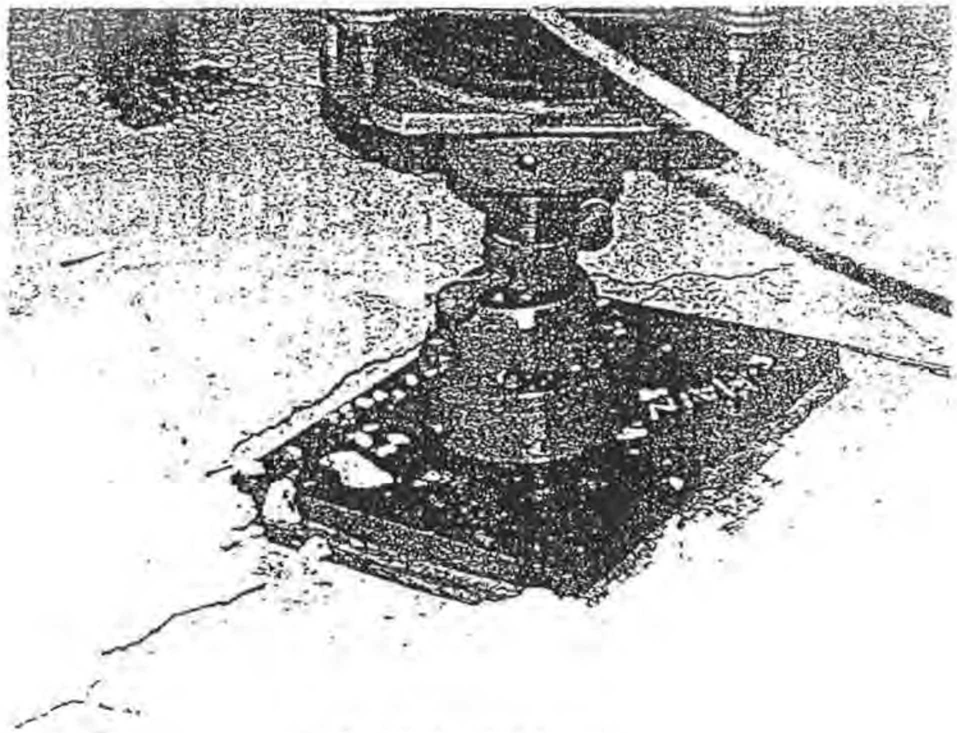
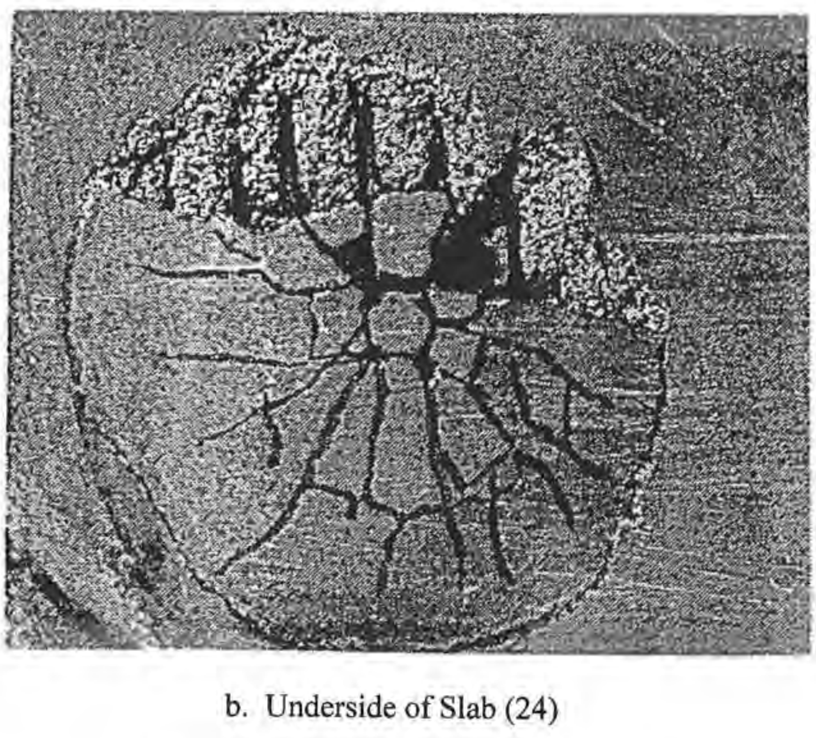


Figure 2.16. Punching Shear Failure Mode (11)



a. Top Side of Slab (11)



b. Underside of Slab (24)

Figure 2.17. Punching Shear Failure

Deck Stress State. Truck traffic/loadings by far cause the greater service load stresses and strains in bridge decks and do the greatest damage. At the location of the truck tires, the deck deflects in somewhat of a dish shape as indicated in Figure 2.18. Thus, slightly away from the tire, the biaxial bending compressive stresses in the top of the deck are actually a less damaging load state than would exist for a uniaxial bending as can be seen from the biaxial concrete failure envelope shown in Fig. 2.19. Also note in Fig. 2.19, that the biaxial tensile stresses on the bottom of the deck are no more damaging than those caused by uniaxial bending. Immediately under the truck wheel load, where the stress state is triaxial, the deck stress state is even less damaging as indicated in Fig. 2.20.

Deck Fatigue. Barker and Puckett (6) in their book on Design of Highway Bridges have this to say about fatigue of concrete flexural members.

> When plain concrete beams or slabs are subjected to repetitive stresses that are less than the static strength, accumulated damage due to progressive internal microcracking eventually results in a fatigue failure. If the repetitive stress level is decreased, the number of cycles to failure N increases. This effect is shown by the S-N curves in Figure 2.21, where the ordinate is the ratio of the maximum stress S_{max} to the static strength and the abscissa is the number of cycles to failure N, plotted on a logarithmic scale. For the case of plain concrete beams, S_{max} is the tensile stress calculated at the extreme fiber assuming an uncracked section and the static strength is the rupture modulus stress f_r.
>
> The curves a and c in Figure 2.21 were obtained from tests in which the stress range between a maximum stress and a minimum stress equal to 75% and 15% of the maximum stress, respectively. It can be observed that an increase of the stress range results in a decreased fatigue strength for a given number of cycles. Curves b and d indicate the amount of scatter in the test data. Curve b corresponds to an 80% chance of failure while curve d represents a 5% chance of failure. Curves a and c are averages representing 50% probability of failure.
>
> The S-N curves for concrete in Figure 2.21 are nearly linear from 100 cycles to 10 million cycles and have not flattened out at the higher number of cycles to failure. It appears that concrete does not exhibit a limiting value of stress below which the fatigue life is infinite. Thus, any statement on the fatigue strength of concrete must be given with reference to the number of cycles to failure. ACI Committee 215 (1992) concludes that the fatigue strength of concrete for the life of 10 million cycles of load and a probability of failure of 50%, regardless of whether the specimen is loaded in compression, tension, or flexure, is approximately 55% of the static strength.

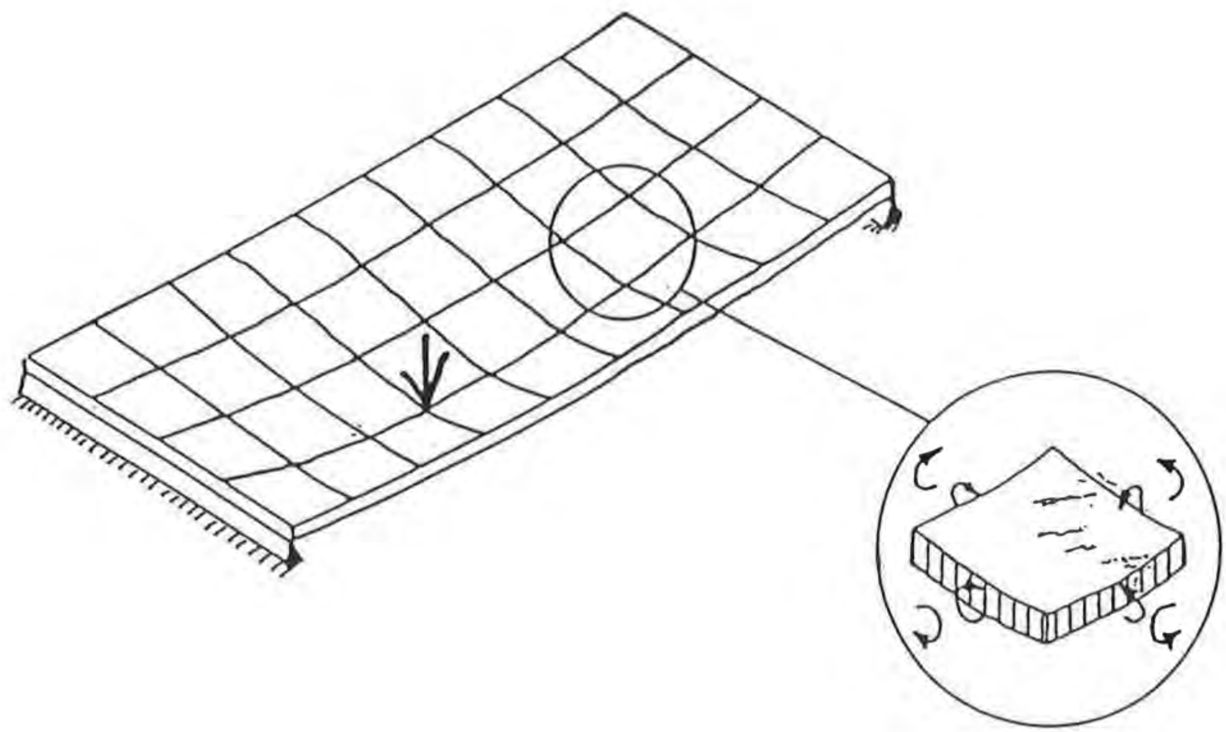


Fig. 2.18 Bridge Deck Deformations and Moments From a Truck Tire Loading (16)

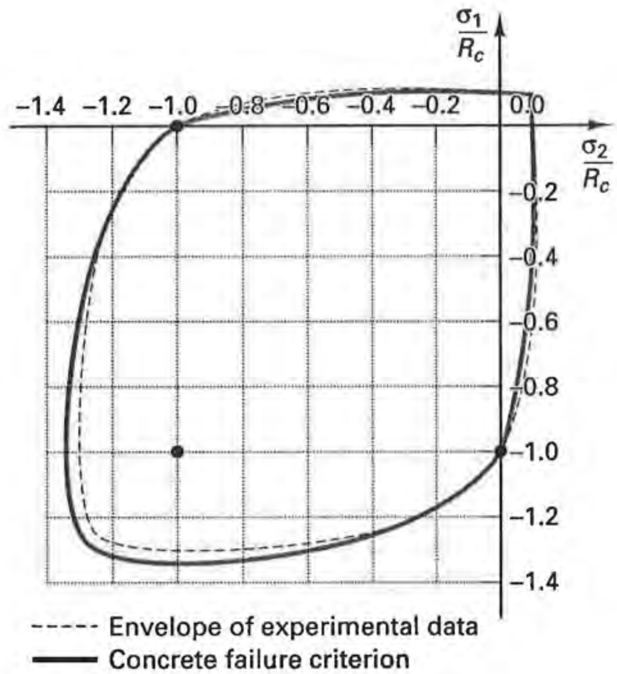
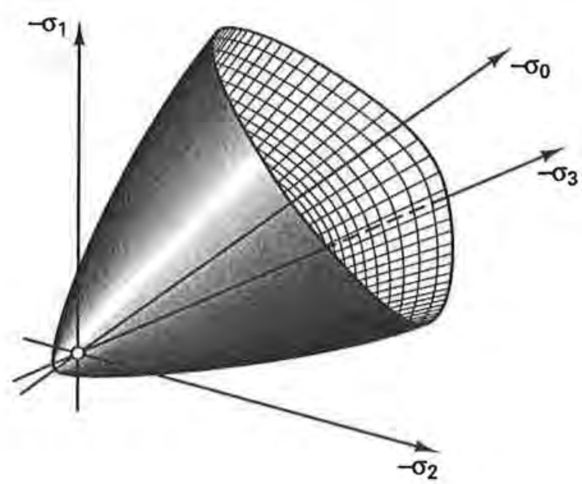


Fig. 2.19 Concrete Biaxial Failure Predictions (23)



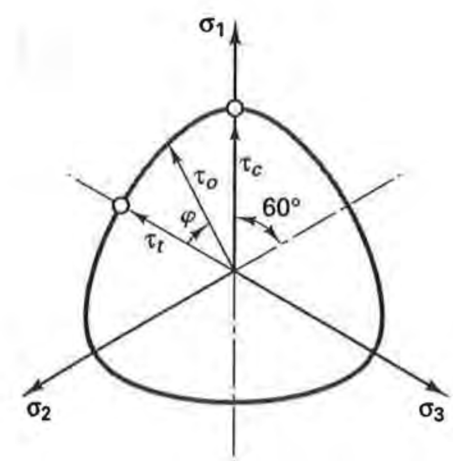


Fig. 2.20 Concrete Triaxial Failure Surface (23)

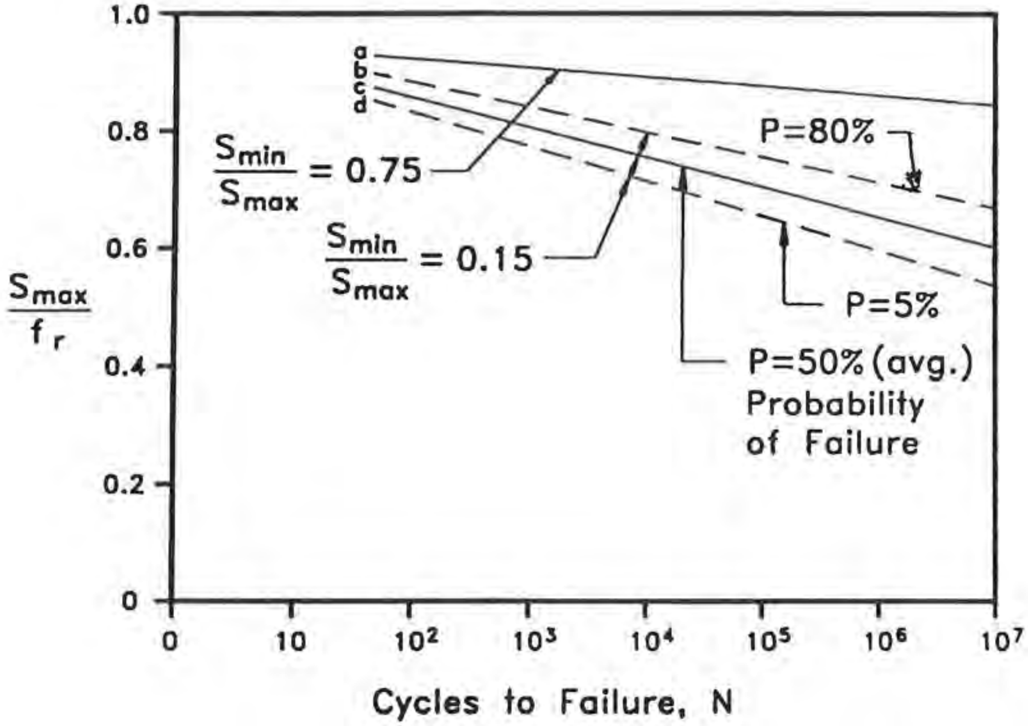


Fig. 2.21. Fatigue Strength of Plain Concrete Beams (6)

Table 2.5. Section Modulus Per Unit Width for Slabs 6 to 9 Inches Thick

D (in)	D^2 (in)	$D^2/D^2_{6"}$
6	36	1.00
7	49	1.36
8	64	1.78
9	81	2.25

Beam on elastic foundation theory indicates slab bending moment values well in excess of those necessary to crack the slab concrete. With repeated loading, and with a fatigue strength of approximately 55% of the static strength (at 10 MC of loading), even more slab cracking will occur due to fatigue. This will cause a poorer distribution of loading to the girders, and in-turn reduces the girder fatigue/service life. It will also cause more movement and working of the existing slab cracks, and thus a lengthening and widening of these cracks. This will result in a more rapid deterioration of the slab due to mechanical actions of abrasion, greater wheel impact, debris entering the wider cracks, etc. It will also allow greater damage due to water and chloride ions entering the slab via these cracks.

Since slab cracking due to bending varies inversely with the section modulus, it thus varies inversely with the square of the slab thickness, D. Table 2.5 shows how D^2 and a normalized (with respect to a 6" slab) D^2 vary for slab thickness of 6 to 9 inches. Obviously employing thicker slabs would reduce slab static load cracking as well as later fatigue load cracking. In-turn, this will enhance the service life of the slab. Also, thicker decks would reduce slab rebar stresses and enhance their fatigue life. However, such enhancements are not needed, as the writer is not aware of any instances of slab rebar fatigue failure.

Batchelor, et al. (7) reported, based on their study using 1/8 - scale deck slab models, that deck slabs of conventional design (AASHTO design) have large reserves of fatigue strength. They indicated that an endurance limit of 50% of the ultimate capacity can be expected in such deck slabs, and an endurance limit of 40 % can be expected in slabs with less reinforcement, i.e., 0.2% isotropic reinforcement, and even in unreinforced slabs.

Code Minimum Deck Thickness Requirements. Current AASHTO requirements for the minimum thickness of concrete bridge decks is 175 mm (approximately 7 in) superimposed on the traditional minimum depth of slab based on the deck span length S to control deflections. The traditional minimum depth equation used was

$$h_{\min} = \frac{S+3000}{30} \tag{2.6}$$

where S and h are in mm.

The new requirement limits h_{min} to 175 mm and thus Equation (2.6) is modified to

$$h_{\min} = \frac{S+3000}{30} \tag{2.7}$$

$$\geq 175 \text{ mm}$$

Or alternatively,

$$S_{\max} = 30h - 3000 \tag{2.8}$$

Using Eqn. (2.8) for various deck thicknesses, h, yields the maximum girder spacings, S, and maximum $\frac{S}{h}$ ratio values shown in Table 2.6. These values are also shown plotted in Fig. 2.22.

The ACI (2) recommends a minimum thickness of 8" for bridge decks. The AASHTO and ACI minimum deck thickness requirements are summarized in graphical form in Fig. 2.23.

Zia, et al. (36) wrote Eqn. (2.8) in mixed English units as

$$S_{\max} = h(\frac{30}{12}) - 10 \tag{2.9}$$

where S = girder spacing in ft.

h = deck thickness in in.

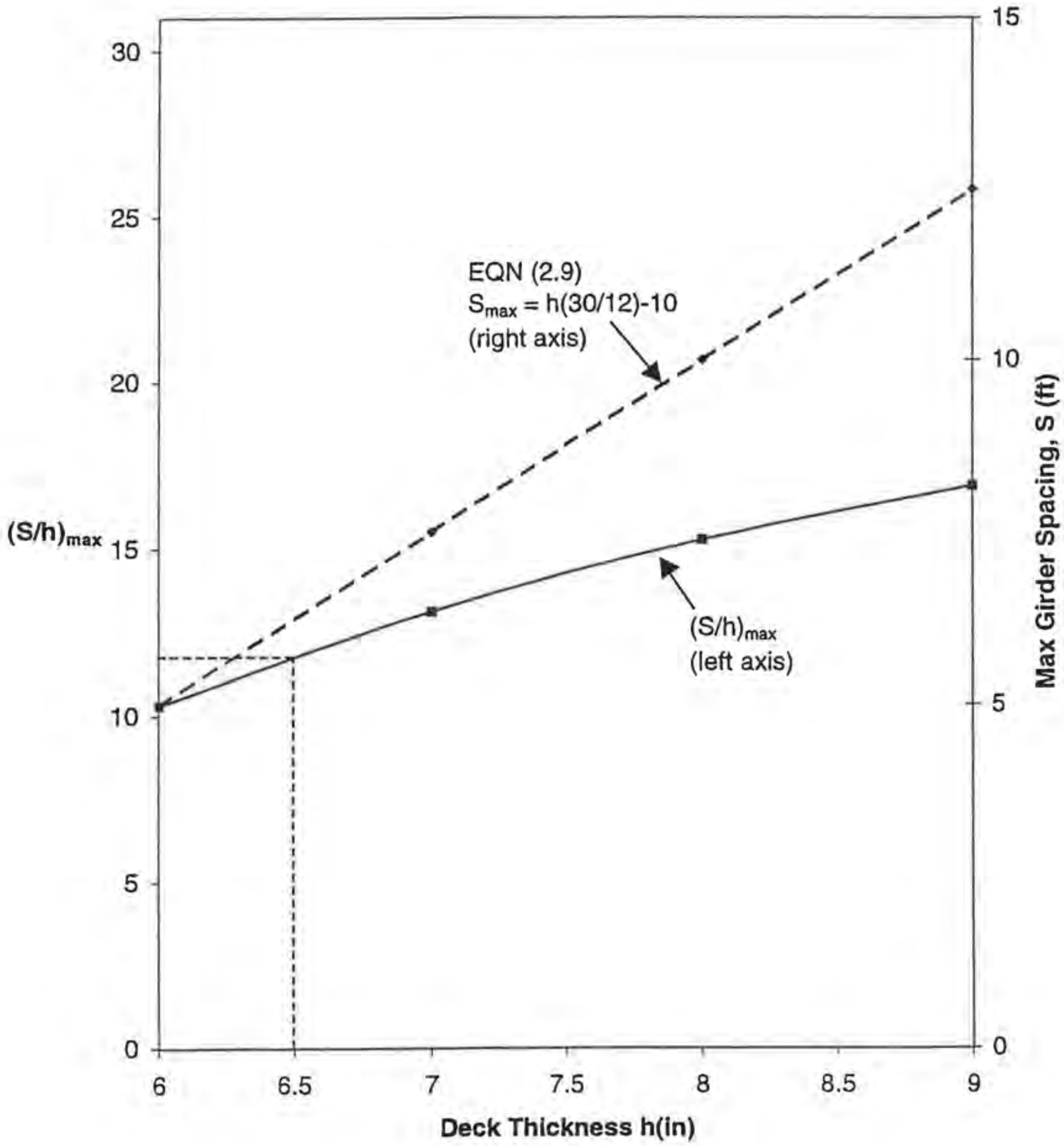


Fig. 2.22 AASHTO Maximum Girder Spacing and S/h Ratio vs. Deck Thickness

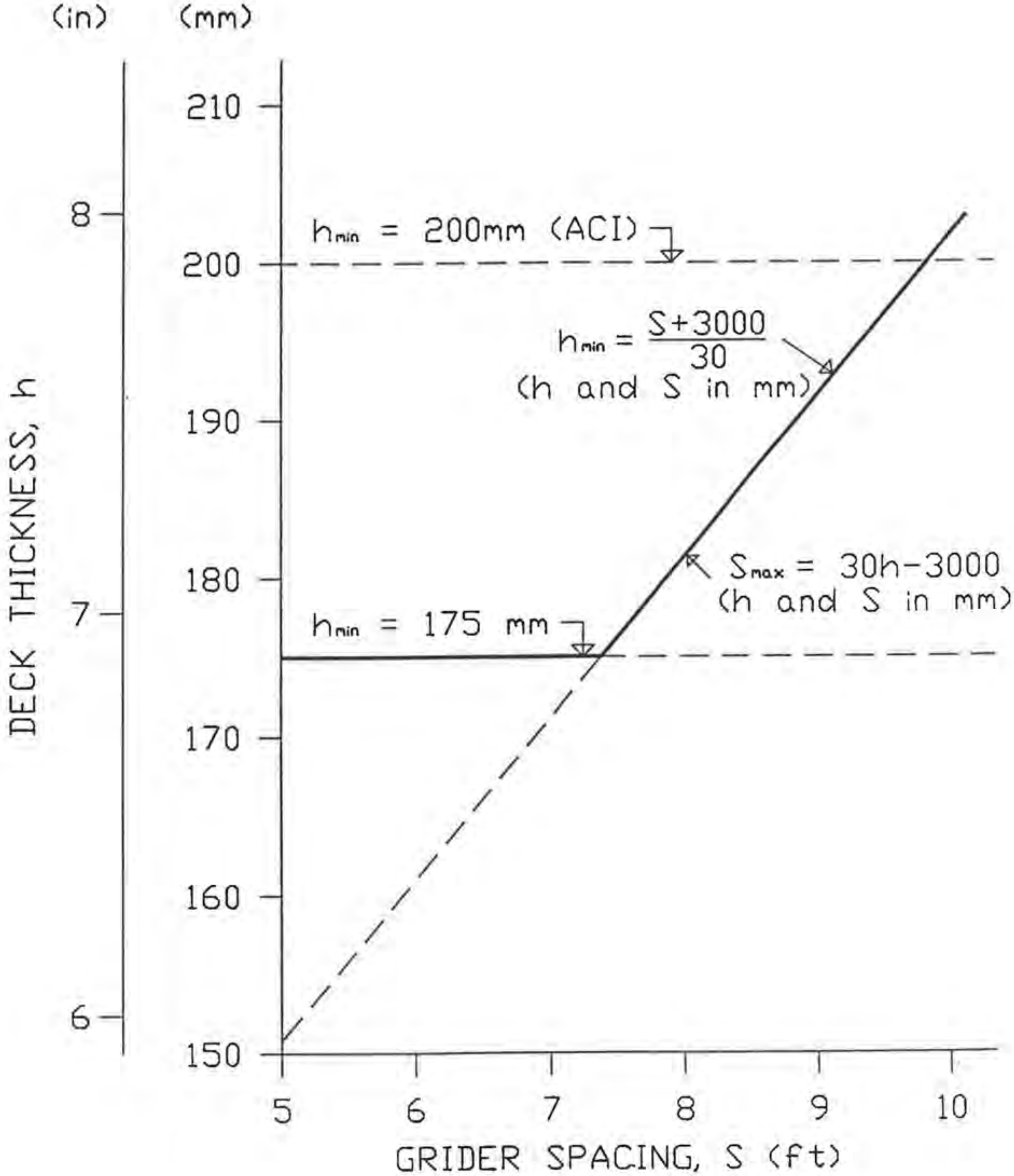


Fig. 2.23 AASHTO Requirements for Minimum Bridge Deck Thickness

They used this equation to compute the maximum girder spacing for various deck thicknesses. They also computed the maximum girder spacing based on strength criteria for different deck concrete strengths. These are shown in Table 2.7, and it can be seen that the deflection criteria, rather than the strength criteria, controls the maximum girder spacing. Equation (2.9) is also plotted in Fig. 2.22.

ACI 343R-95 (1) indicates that it is generally more economical to use a thicker deck slab with a wider girder spacing, and to cantilever the slab beyond the exterior girder 1/3 to ½ of the girder spacing. They recommend as a design guideline the effective deck span - deck thickness values shown in Table 2.8 for an AASHTO HS20 design loading.

It should be noted that in the 1995 UAB study of deterioration of bridge decks ($D \approx 6.5$") in Birmingham, AL (9), four out of five undamaged decks in that study has S/D ratios less than the 12.1 maximum value shown in Fig. 2.22 (see dotted line), and four out of five of the damaged decks in that study had S/D ratios greater than the 12.1 maximum value.

Ramey, et al. (26) point out that uniformity of deck thickness at 8" would allow standardization of rebar mat support chairs in attaining proper bottom and top covers, and would allow the use of a 2½" topbar cover if desired. It would simplify construction and inspection, and should in turn result in enhanced deck quality/durability. Uniformity of deck thickness would also allow ALDOT engineers and inspectors to make better estimates of span deflections due to placement of the fresh concrete. Even though span lengths will continue to vary, having the same concrete uniform DL on the bridges each time will remove one of the variables and should result in more accurate defections assessments due to concrete DL. This is important in attaining the proper deck thickness and cover on the deck top bars during the construction process, and in turn on minimizing deck transverse cracking and maximizing deck durability.

Table 2.6. AASHTO Max. Girder Spacing and S/h Ratio Values Based on Deflections

Deck Depth, h (in)	Maximum Girder Spacing, S, (ft)	Maximum (S/h)
6.0	5.15	10.32
7.0	7.66	13.13
8.0	10.16	15.26
9.0	12.66	16.88

Table 2.7 Max. Girder Spacing for Various Deck Thicknesses and Concrete Strengths (36)

Deck Thickness (in)	Maximum Girder Spacing, S (ft)			
	Deck Concrete Strength[1] (psi)			Eq. 2.9[2]
	3,000	4,000	5,000	
6	4.87	7.04	8.32	5.00
7	8.77	12.18	14.51	7.50
8	12.92	17.17	20.59	10.00
9	18.06	23.08	25.88	12.50
10	21.86	28.08	31.58	15.00

[1]Girder spacing based on strength criteria
[2]Girder spacing based on deflection criteria (rounded-off)

Table 2.8 Recommended Guide for Bridge Deck Thickness for Various Effective Spans (1)

Effective span length*		Slab thickness, h	
Ft	m	in.	mm
6	1.8	7¼	185
7	2.1	7¼	195
8	2.4	8	205
9	2.7	8½	215
10	3.0	8¾	220
11	3.4	9	230
12	3.7	9½	240
13	4.0	9¾	250
14	4.3	10	260
15	4.6	10½	270
16	4.9	11	280

*Effective span is the clear distance between webs of T-beams or box girders or the top flanges of I-beams.

Deck Thickness Changes of Other Agencies. In 1988 the New Jersey DOT increased their bridge deck thickness on all new decks, and increased the minimum deck thickness to 8" (26). They have since observed less deck cracking and feel that they now have a very good and durable deck design.

Around 1970, the Texas DOT instituted several bridge deck design changes in an effort to mitigate their deck deterioration problem (5). One of their primary changes was to increase deck thickness from 6" to typically 7" - 8". In addition to increasing the deck thickness by 1^+ inches, they also reduced their allowable LL deflection for bridges. The Texas DOT's current philosophy (5) seems to be that the stiffer the bridge deck, the greater is its durability.

A recent study conducted in Japan (26) determined that the primary deterioration mechanism for bridge decks in that country was

(1) development of early penetrating cracks due to shrinkage (drying and thermal)

(2) infiltration and leakage of rainwater through these cracks

(3) abrasive actions caused by working the cracks in shear in transmitting wheel loads.

They concluded that the corrective actions required were:

(1) eliminate or minimize early penetration shrinkage cracks

(2) seal cracks to prevent infiltration of water

(3) use more shrinkage/temperature steel to control crack width, and increase deck thickness to increase shear capacity and minimize relative movement across the crack. Japan increased their deck thickness from approximately 7" to 9".

Methods of Increasing Live Load Capacity. Six methods for increasing live load capacity of highway bridges have been used for many years and can be considered standard methods. These were identified in Ref. (18), and are:

- Reduce dead load,
- Develop composite action,
- Increase transverse stiffness,
- Improve member strength,
- Post-tension members, and
- Develop continuity.

These same methods would also be effective in reducing LL stresses and thus extending the fatigue life of bridge components. For beam and slab bridges, increasing the transverse stiffness improves the load distribution to the beams and enables a higher live load to be applied, or a greater number of cycles of LL before reaching the fatigue limit.

Increased transverse stiffness can be obtained by thickening the deck slab. The Michigan DOT reported on casting a concrete slab over an existing slab on a concrete T-beam bridge to upgrade its capacity. After testing the strengthened bridge, a 32 percent increase in live load was permitted. This could be an economical method for relatively short spans, as no special deck forms are needed. The Michigan project has spans of 42 ft. or less; however, longer spans would result in an increasing dead load penalty (13).

Although the above methods were identified and are used for existing bridges, they also indirectly speak to the effectiveness of employing thicker decks in new designs to improve deck and superstructure strength and stiffness, and to reduce LL stresses, strains, and cracking. In turn this should enhance the deck and superstructure fatigue life.

3. RESULTS OF CIP REINFORCED CONCRETE BRIDGE DECK THICKNESS SURVEY

3.1 General

In an effort to identify the required design values for deck thickness and related parameters for CIP reinforced concrete bridge decks used by other highway agencies in the United States, a relatively short survey questionnaire was prepared. The survey questions were fairly broad with the plan being that once the broad picture was clear, the PI could contact the appropriate states for further details where needed. A copy of the survey questionnaire is shown in Appendix A. The questionnaire was sent to the State Bridge Engineer in each of the 50 states, Puerto Rico, and the New York State Thruway Authority during the first week of March 1999, with a requested return date of April 5, 1999. Forty-four (44) agencies graciously participated in the survey for an 85 percent response. Results of the survey are presented in Sections 3.2 and 3.3 below.

3.2 Results of State DOT Survey

A copy of the survey questionnaire is included below to serve to,

1. make known the questions asked,
2. summarize some of the survey results,
3. provide a cross-reference of where the survey results are presented.

Survey Questionnaire
on
CIP Reinforced Concrete Bridge
Deck Thickness

Instructions

It is recognized that poor construction practices can negate the best of designs and lead to inferior bridge decks. However, the focus of this questionnaire is on design parameter criteria to enhance the serviceability performance of bridge decks. The questions pertain to required design values for deck thickness and related parameters for CIP reinforced concrete bridge decks. The Alabama Department of Transportation (ALDOT) has a "Bridge Deck Structured Design Criteria" summary figure depicting most of the deck design parameter values for different girder spacing. If your DOT has a similar summary figure would you please enclose a copy as part of your response to this questionnaire.

Questions

1. If your DOT has a deck structural design summary figure, will you please share it with us via attaching it to this questionnaire. If you were able to provide a summary figure, proceed to Question 4. If not, proceed to the next question.

   N/A here

2. What deck thickness do you typically use for the following girder spacings?

Girder Spacing	6'	7'	8'	9'	10'
Deck Thickness					

   See Table 3.2

3. What deck top and bottom rebar clearance do you call for? CL_{top} = CL_{bottom} = See Fig 3.4 and Table 3.1

4. Do you use deicing salt on your decks as needed during the winter season?
   ☐ Yes ☐ No

   Yes - 93%
   No - 7%

5. Do you design-in an extra 1/2" (or other) to the deck thickness (above the top mat) to allow for possible future grinding for rideability? ☐ Yes ☐ No

   Yes - 61%
   No - 39%

6. What strength/grade deck concrete and reinforcing steel do you call for?
   f'_c =
   F_y =
   See Fig 3.3 ad Table 3.1

7. Do you require, or typically employ, a wearing surface on your bridge decks?
   ☐ Yes ☐ No

   Yes - 33%
   No - 67%

   If yes, what wearing surface material and thickness do you typically use?

   See Fig. 3.2 and Table 3.1

8. What LL deflection requirement do you impose for your bridge superstructures?

See Table 3.1

9. It is reported in the literature that many state DOTs enacted deck design changes of increasing deck thicknesses (typically from 6" - 6½" to 8" - 8½"), and increasing cover on the top mat steel in the 1970s. Did your state make such changes?
☐ Yes ☐ No

Yes - 71%
No - 29%

If yes, has there been any significant improvements in performance and/or service life due to increasing the deck thickness?

See Appendix A

10. Have you made any significant changes (other than those in the previous question if applicable) in your deck thickness and/or rebar cover requirements in the last 30 or so years? ☐ Yes ☐ No

Yes - 49%
No - 51%

If yes, what were these changes, and how would you assess the results of the changes?

See Appendix A

11. Do you believe that excessive deck cracking and premature deterioration are related to deck thickness? ☐ Yes ☐ No

Yes - 58%
No - 42%

If yes, in what way, and do you have any data or performance information to support your belief?

See Appendix A

12. Do you believe that excessive deck cracking and premature deterioration are related to bridge superstructure flexibility? ☐ Yes ☐ No

Yes - 64%
No - 36%

If yes, in what way, and do you have any data or performance information to support your belief?

See Appendix A

13. Do you believe it would be a good idea to use a "standard" (with exceptions allowed) deck thickness (of say 8"), and to vary girder spacing and the reinforcing steel as necessary in order to more closely "<u>standardize" deck construction (and inspection) practices,</u> and in turn enhance the quality of the as-built decks?
☐ Good Idea ☐ Bad Idea

Good Idea - 65%
Bad Idea - 35%

Why?

See Appendix A

14. Please list any <u>design actions</u> you would suggest to enhance the serviceability/service life of CIP reinforced concrete bridge decks.

See Appendix A

Table 3.1 provides a rather comprehensive summary regarding deck top and bottom rebar clearance, environmental exposure conditions, strength of concrete and steel used, wearing surfaces, live load deflection requirements, changes in design parameter over the last 30 years, and opinions on the relationship of deck thickness and superstructure flexibility with deck cracking and premature deterioration. Figure 3.1 summarizes the state responses to the eight "yes/no" survey questions (these results are also shown on the questionnaire). This figure reflects that for the participating agencies, almost all (93%) use deicing salt on their decks as needed during the winter season. Over half (61 %) design-in an extra ½" (or other) to the deck thickness (above the top mat) to allow for possible future grinding for rideability and/or skid resistance. One-third (33%) require, or typically employ, a wearing surface on their bridge decks. Seventy-one percent stated that they had increased deck thickness and cover on the top mat in the 1970s. Half (49%) of the agencies made other significant changes in deck thickness and/or rebar cover requirements in the last 30 years. Fifty-eight percent believe that excessive deck cracking and premature deterioration are related to deck thickness, and sixty-four percent believe that excessive deck cracking and premature deterioration are related to bridge superstructure flexibility. Approximately two-out-of-three (65%) believe that it is a good idea to use a "standard" (with exceptions allowed) deck thickness and to vary girder spacing and the reinforcing steel as necessary in order to more closely "standardize" deck construction and inspection practices, and in turn enhance the quality of as-built decks.

Table 3.2 reflects bridge deck thicknesses of all of the responding states over a range of girder spacings of 6 - 10 ft. The thicknesses range from a low of 7" for Alabama to a high of 9 ½" for New

York. As can be seen in Table 3.2, Alabama's typical deck thickness of 7" is significantly lower than the other states. Also as can be seen in that table, 8" is probably the mode of the thicknesses, and numerous states employ a constant deck thickness over the range of girder spacings shown.

Figure 3.2 summarizes the type of wearing surface materials used by the agencies that require a wearing surface. Asphalt with membrane was by far the material of choice, with 11 of the 14 states (79%) employing a wearing surface using this material in thicknesses ranging from ½" to 3⅛" with 3" being the mode of thickness employed. PCC integral with the deck, low slump concrete, and latex modified concrete overlay, were each reported to be used by 1 state (7% each) to account for the remaining 21 % of the agencies using a wearing surface on bridge decks.

Figure 3.3 shows that most states (42%) employ 4.0 ksi concrete for their bridge decks, as does Alabama, and 77% of the states employ 4.0 ksi or higher deck concrete. Figure 3.4 reflects that most states (55%) employ a top bar cover of 2.5 in. on their decks with 2.0 in. being the second most widely used (26%) top cover. Alabama is in this latter group. Nearly 2-in-3 states (64%) use a bottom bar cover of 1.0 in., with 1.5 in. being the second most widely used (26%) bottom cover.

A summary of pertinent comments made by the survey responders to questions on (1) deck cracking and premature deterioration being related to deck thickness and/or superstructure flexibility, (2) changes made in the last 30 years in deck thickness and/or rebar cover requirements, and (3) other design actions recommended to enhance deck service life is given in Appendix A. Also given Appendix A is a listing of pertinent comments made in response to the survey question on using a "standard" bridge deck thickness.

3.3 Southern States Survey Results

Of greater interest than how ALDOT bridge deck thicknesses, rebar cover, and deck concrete strength compares with other states in the U.S., is how they compare with other states in the south since the environmental exposure conditions are more similar for these states. Thus, a subset of the deck data from the survey for the ten southern states from Texas to the Carolinas was extracted and is shown in Table 3.3. The following observations can be made from these data.

1. Eight of the ten southern states use deicing salts on their bridge decks. Only Florida and Alabama do not.

2. Only Florida and North Carolina of the ten southern states require or employ a wearing surface on their bridge decks.

3. Five (or half) of the states use a top bar cover in excess of 2" (around 2 ½"), and five (or half) use a 2" top bar cover.

4. Seven of the ten states use a bottom bar cover of 1". Florida used a large value of 2" due to many of its bridges being in a splash zone of seawater.

5. Five states use deck concrete having a compressive strength of 4.0 ksi. Florida used a higher value of 4.5 ksi to try to reduce concrete permeability and thus reduce rebar corrosion. The remaining four states use deck concrete strengths of 3.0-3.5 ksi.

6. All of the southern states use thicker decks than Alabama. The additional thickness varies from about ½" - 1 ½".

7. For the common girder spacings of 6' - 8', the typical thickness (excluding Alabama) is 8" while Alabama is 7". Thus on average, Alabama bridge decks are approximately 1" thinner than those of its sister states in the south.

3.4 Closure

Information gleaned from the mail questionnaire survey of all the state DOT's (85% of the states responded) in the U.S. that is of greatest interest to this investigation is as follows:

1. Alabama has the thinnest bridge decks in the U.S.

2. The range of deck thicknesses in the U.S. (for typical girder spacings of 6'-9') is 7" (Alabama) to 9.5" (New York), with the mode being 8".

3. Most states (61%) design-in an extra ½" (or other) to the deck thickness above the top mat to allow for future grinding for improved rideability and/or skid resistance.

4. Most states (55%) employ a top bar cover of 2.5", and 64% employ a bottom bar cover of 1".

5. Most states (67%) do not employ a wearing surface on their bridge decks.

6. Most states (77%) employ concrete strength in the range of 4.0-4.5 ksi for their bridge decks, and 44% of the states use 4.0 ksi concrete.

7. Most states believe that excessive deck cracking and premature deterioration are related to deck thickness (58%), and are related to superstructure flexibility (64%).

8. Most states (65%) believe that it is a good idea to use a "standard" deck thickness (with exceptions allowed) in order to enhance deck quality and durability through greater standardization in construction and inspection, and half of the states (50%) are currently employing this practice.

9. Half (5) of the southern states use a deck top bar cover of 2", and the other half (5) use a cover in excess of 2" (around 2 ½").

10. All of the southern states use thicker decks than Alabama, with the additional thickness varying from about ½" - 1½". For the common girder spacings of 6' - 8', the typical thickness of our sister southern states is 8", while Alabama's is 7".

Table 3.1 Summary of Survey Questionnaire Responses from State DOT's on CIP Reinforced Concrete Bridge Deck Thickness.

Question/Area of Inquiry	Alaska	Arizona	Arkansas	California	Colorado
Deck top and bottom rebar clearance	top = 2½" bottom = 1"	top = 2½" bottom = 1"	top = 2" bottom = 1"	top = 2" bottom = 1"	top = 2½" bottom = 1"
Use deicing salt on decks	Yes	Yes	Yes	Yes	Yes
Design-in an extra ½" (or other) to the deck thickness (above top mat)	Yes	Yes	Yes	–	Yes
Strength/grade deck concrete and reinforcing steel	fc = 3.0 ksi to 3.5 ksi Fy = 60 ksi	fc = 4.5 ksi Fy = 60 ksi	fc = 4.0 ksi Fy = 60 ksi	fc = 3.25 ksi Fy = 60 ksi	fc = 4.5 Fy = 60 ksi
Require or employ wearing surface	Yes	No	No	No	Yes
Wearing surface material and thickness used	2 ½" to 3" asphalt with membrane	–	–	–	2" Asphalt over waterproofing membrane
Live load deflection requirements for bridge superstructure	AASHTO	AASHTO	AASHTO	L/800 L/1000 L/300	L/800
Increased deck thickness and cover on the top mat of steel in the 1970's	Yes	Yes	No	Yes	No
Any other significant changes in deck thickness and/or rebar cover in last 30 years	No	Yes	No	No	No
Believe that excessive deck cracking and premature deterioration related to deck thickness	No	Yes	No	–	Yes
Believe that excessive deck cracking and premature deterioration related to superstructure flexibility	No	Yes	Yes	–	Yes
Good idea to use a standard deck thickness and vary girder spacing and reinforcing steel as necessary to standardize deck construction and in turn enhance the quality of as-built decks	Bad Idea	Bad Idea	Good Idea	Bad Idea	–

Table 3.1 Summary of Survey Questionnaire Responses from State DOT's on CIP Reinforced Concrete Bridge Deck Thickness

Question/Area of Inquiry	Connecticut	Delaware	Florida	Georgia	Idaho
Deck top and bottom rebar clearance	top = 2" bottom = 1"	top = 2" bottom = 1"	top = 2" bottom = 1"	top = 2¼" or 2¾" bottom = 1"	top = 2½" bottom = 1½"
Use deicing salt on decks	Yes	Yes	No	Yes	Yes
Design-in an extra ½" (or other) to the deck thickness (above top mat)	No	No	Yes	Yes	Yes
Strength/grade deck concrete and reinforcing steel	fc =4 ksi Fy = 60 ksi	fc = 4.5 ksi Fy = 60 ksi	fc = 4.5 ksi Fy = 60 ksi	fc = 3.0 ksi or 3.5 ksi Fy = 60 ksi	fc = 4.0 Fy = 60 ksi
Require or employ wearing surface	Yes	No	Yes	No	No
Wearing surface material and thickness used	2 $^{9}/_{16}$" bituminous concrete on waterproofing membrane	–	–	–	–
Live load deflection requirements for bridge superstructure	AASHTO	AASHTO	AASHTO	L/800 L/1000	–
Increased deck thickness and cover on the top mat of steel in the 1970's	No	No	Yes	Yes	Yes
Any other significant changes in deck thickness and/or rebar cover in last 30 years	No	Yes	Yes	Yes	Yes
Believe that excessive deck cracking and premature deterioration related to deck thickness	Yes	No	No	Yes	Yes
Believe that excessive deck cracking and premature deterioration related to superstructure flexibility	No	No	–	Yes	–
Good idea to use a standard deck thickness and vary girder spacing and reinforcing steel as necessary to standardize deck construction and in turn enhance the quality of as-built decks	Good Idea	Bad Idea	Good Idea	–	Good Idea

Table 3.1 Summary of Survey Questionnaire Responses from State DOT's on CIP Reinforced Concrete Bridge Deck Thickness

Question/Area of Inquiry	Illinois	Iowa	Kentucky	Louisiana	Maine
Deck top and bottom rebar clearance	top = 2¼" bottom = 1"	top = 2½" bottom = 1"	top = 2" bottom = 1"	top = 2" bottom = 1"	top = 2" bottom = 1"
Use deicing salt on decks	Yes	Yes	Yes	Yes	Yes
Design-in an extra ½" (or other) to the deck thickness (above top mat)	No	No	No	Yes	No
Strength/grade deck concrete and reinforcing steel	fc = 3.5 ksi Fy = 60 ksi	fc = 4.0 ksi Fy = 60 ksi	fc = 4.0 ksi Fy = 60 ksi	fc = 3.2 ksi Fy = 60 ksi	fc = 4.35 Fy = 60 ksi
Require or employ wearing surface	No	Yes	No	No	Yes
Wearing surface material and thickness used	–	1/2" PCC integral with deck	–	–	3" bituminous with waterproofing membrane
Live load deflection requirements for bridge superstructure	AASHTO	AASHTO	AASHTO	AASHTO	AASHTO
Increased deck thickness and cover on the top mat of steel in the 1970's	Yes	–	Yes	Yes	Yes
Any other significant changes in deck thickness and/or rebar cover in last 30 years	Yes	No	Yes	No	No
Believe that excessive deck cracking and premature deterioration related to deck thickness	No	Yes	Yes	No	Yes
Believe that excessive deck cracking and premature deterioration related to superstructure flexibility	No	No	Yes	Yes	–
Good idea to use a standard deck thickness and vary girder spacing and reinforcing steel as necessary to standardize deck construction and in turn enhance the quality of as-built decks	–	Good Idea	Good Idea	Bad Idea	Bad Idea

Table 3.1 Summary of Survey Questionnaire Responses from State DOT's on CIP Reinforced Concrete Bridge Deck Thickness

Question/Area of Inquiry	Maryland	Massachusetts	Michigan	Minnesota	Mississippi
Deck top and bottom rebar clearance	top = 2½" bottom = 1"	top = 2" bottom = 1½"	top = 3" bottom = 1½"	top = 3" bottom = 1"	top = 2¼" bottom = 1"
Use deicing salt on decks	Yes	Yes	Yes	Yes	Yes
Design-in an extra ½" (or other) to the deck thickness (above top mat)	Yes	No	No	No	No
Strength/grade deck concrete and reinforcing steel	fc = 4.5 ksi Fy = 60 ksi	fc = 4.35 ksi Fy = 60 ksi	fc = 4.5 ksi Fy = 60 ksi	fc = 4.0 ksi Fy = 60 ksi	fc = 4.0 Fy = 60 ksi
Require or employ wearing surface	No	Yes	No	Yes	No
Wearing surface material and thickness used	–	3 ⅛" bituminous concrete over waterproofing membrane	–	2" low slump concrete	–
Live load deflection requirements for bridge superstructure	AASHTO	L/800 L/1000	L/800	L/800 L/1200	–
Increased deck thickness and cover on the top mat of steel in the 1970's	Yes	No	Yes	Yes	No
Any other significant changes in deck thickness and/or rebar cover in last 30 years	Yes	Yes	Yes	Yes	No
Believe that excessive deck cracking and premature deterioration related to deck thickness	Yes	Yes	No	Yes	No
Believe that excessive deck cracking and premature deterioration related to superstructure flexibility	Yes	Yes	Yes	Yes	No
Good idea to use a standard deck thickness and vary girder spacing and reinforcing steel as necessary to standardize deck construction and in turn enhance the quality of as-built decks	Good Idea	Good Idea	Good Idea	Good Idea	Good Idea

Table 3.1 Summary of Survey Questionnaire Responses from State DOT's on CIP Reinforced Concrete Bridge Deck Thickness

Question/Area of Inquiry	Missouri	Montana	Nebraska	Nevada	New Hampshire
Deck top and bottom rebar clearance	top = 2¾" bottom = 1"	top = 2½" bottom = 1"	top = 2½" bottom = 1"	top = 2½" bottom = 1"	top = 2½" bottom = 1¼"
Use deicing salt on decks	Yes	Yes	Yes	Yes	Yes
Design-in an extra ½" (or other) to the deck thickness (above top mat)	Yes	Yes	No	Yes	Yes
Strength/grade deck concrete and reinforcing steel	fc = 4.0 ksi Fy = 60 ksi	fc = 4.0 ksi to 4.5 ksi Fy = 60 ksi	fc = 4.0 ksi Fy = 60 ksi	fc = 4.0 ksi to 4.5 ksi Fy = 60 ksi	fc = 4.0 Fy = 60 ksi
Require or employ wearing surface	No	No	No	No	Yes
Wearing surface material and thickness used	–	–	–	–	barrier membrane plus two 1" layers of asphalt pavement
Live load deflection requirements for bridge superstructure	L/1000	L/1000	AASHTO	AASHTO	L/1200 L/1600
Increased deck thickness and cover on the top mat of steel in the 1970's	Yes	Yes	No	No	Yes
Any other significant changes in deck thickness and/or rebar cover in last 30 years	No	No	No	Yes	Yes
Believe that excessive deck cracking and premature deterioration related to deck thickness	Yes	No	_	_	No
Believe that excessive deck cracking and premature deterioration related to superstructure flexibility	Yes	Yes	–	–	Yes
Good idea to use a standard deck thickness and vary girder spacing and reinforcing steel as necessary to standardize deck construction and in turn enhance the quality of as-built decks	Good Idea	Bad Idea	Good Idea	Good Idea	Good Idea

Table 3.1 Summary of Survey Questionnaire Responses from State DOT's on CIP Reinforced Concrete Bridge Deck Thickness

Question/Area of Inquiry	New Mexico	New York	New York Thruway Authority	North Carolina	North Dakota
Deck top and bottom rebar clearance	top = 2½" bottom = 1½"	top = 3" bottom = 1½"	top = 3" bottom = 1½"	top = 2½" bottom = 1¼"	top = 2½" bottom = 1"
Use deicing salt on decks	Yes	Yes	Yes	Yes	Yes
Design-in an extra ½" (or other) to the deck thickness (above top mat)	No	Yes	Yes	No	Yes
Strength/grade deck concrete and reinforcing steel	fc = 4.0 ksi Fy = 60 ksi	fc = 3.0 ksi Fy = 60 ksi	fc = 3.0 ksi Fy = 60 ksi	fc = 3.5 ksi Fy = 60 ksi	fc = 4.0 Fy = 60 ksi
Require or employ wearing surface	No	No	Yes	Yes	No
Wearing surface material and thickness used	–	–	7" decks have waterproofing membrane and 2" asphalt overlay	½" asphalt overlay if required in design. Rehabs use latex modified concrete overlay	–
Live load deflection requirements for bridge superstructure	AASHTO	AASHTO	L/800	AASHTO	L/800 L/1000
Increased deck thickness and cover on the top mat of steel in the 1970's	No	Yes	Yes	Yes	Yes
Any other significant changes in deck thickness and/or rebar cover in last 30 years	No	No	Yes	–	Yes
Believe that excessive deck cracking and premature deterioration related to deck thickness	Yes	No	No	Yes	No
Believe that excessive deck cracking and premature deterioration related to superstructure flexibility	Yes	Yes	Yes	Yes	No
Good idea to use a standard deck thickness and vary girder spacing and reinforcing steel as necessary to standardize deck construction and in turn enhance the quality of as-built decks	Bad Idea	Good Idea	Good Idea	Bad Idea	Good Idea

Table 3.1 Summary of Survey Questionnaire Responses from State DOT's on CIP Reinforced Concrete Bridge Deck Thickness

Question/Area of Inquiry	Ohio	Oklahoma	Oregon	Pennsylvania	Puerto Rico
Deck top and bottom rebar clearance	top = 2½" bottom = 1½"	top = 2½" bottom = 1"	top = 2½" bottom = 1¼"	top = 2½" bottom = 1"	top = 2" bottom = 1½"
Use deicing salt on decks	Yes	Yes	No	Yes	No
Design-in an extra ½" (or other) to the deck thickness (above top mat)	Yes	No	Yes	Yes	Yes
Strength/grade deck concrete and reinforcing steel	fc = 4.5 ksi Fy = 60 ksi	fc = 4.0 ksi Fy = 60 ksi	fc = 4.5 ksi Fy = 60 ksi	fc = 4.5 ksi Fy = 60 ksi	fc = 4.0 Fy = 60 ksi
Require or employ wearing surface	_	No	No	Yes	Yes
Wearing surface material and thickness used	_	_	_	_	3" asphalt
Live load deflection requirements for bridge superstructure	AASHTO	L/800 L/1000	AASHTO	AASHTO	AASHTO
Increased deck thickness and cover on the top mat of steel in the 1970's	No	Yes	Yes	Yes	Yes
Any other significant changes in deck thickness and/or rebar cover in last 30 years	Yes	Yes	No	No	No
Believe that excessive deck cracking and premature deterioration related to deck thickness	No	Yes	Yes	No	Yes
Believe that excessive deck cracking and premature deterioration related to superstructure flexibility	No	No	No	Yes	No
Good idea to use a standard deck thickness and vary girder spacing and reinforcing steel as necessary to standardize deck construction and in turn enhance the quality of as-built decks	Bad Idea	Good Idea	Bad Idea	Good Idea	Good Idea

Table 3.1 Summary of Survey Questionnaire Responses from State DOT's on CIP Reinforced Concrete Bridge Deck Thickness

Question/Area of Inquiry	Rhode Island	South Carolina	South Dakota	Tennessee	Texas
Deck top and bottom rebar clearance	top = 2" bottom = 1"	top = 2½" bottom = 1"	top = 2½" bottom = 1"	top = 2½" bottom = 1"	top = 2" bottom = 1½"
Use deicing salt on decks	Yes	Yes	–	Yes	Yes
Design-in an extra ½" (or other) to the deck thickness (above top mat)	No	Yes	Yes	No	No
Strength/grade deck concrete and reinforcing steel	fc = 4.0 ksi to 5.0 ksi Fy = 60 ksi	fc = 4.0 ksi Fy = 60 ksi	fc = 4.5 ksi Fy = 60 ksi	fc = 3.0 ksi Fy = 60 ksi	fc = 4.0 Fy = 60 ksi
Require or employ wearing surface	Yes	No	No	No	No
Wearing surface material and thickness used	1 ½" binder material and 1 ½" Class I-1 bitumen	–	–	–	–
Live load deflection requirements for bridge superstructure	L/800 L/1100	AASHTO	L/1000 L/1200	L/800	AASHTO
Increased deck thickness and cover on the top mat of steel in the 1970's	Yes	Yes	Yes	Yes	No
Any other significant changes in deck thickness and/or rebar cover in last 30 years	Yes	No	No	No	Yes
Believe that excessive deck cracking and premature deterioration related to deck thickness	No	Yes	No	Yes	Yes
Believe that excessive deck cracking and premature deterioration related to superstructure flexibility	Yes	Yes	No	No	Yes
Good idea to use a standard deck thickness and vary girder spacing and reinforcing steel as necessary to standardize deck construction and in turn enhance the quality of as-built decks	Good Idea	Bad Idea	–	Good Idea	Good Idea

Table 3.1 Summary of Survey Questionnaire Responses from State DOT's on CIP Reinforced Concrete Bridge Deck Thickness

Question/Area of Inquiry	Virginia	Wisconsin	Wyoming		Alabama
Deck top and bottom rebar clearance	top = 2¾" bottom = 1½"	top = 2½" bottom = 1½"	top = 2½" bottom = 1"		top = 2" bottom = 1"
Use deicing salt on decks	Yes	Yes	No		No
Design-in an extra ½" (or other) to the deck thickness (above top mat)	Yes	Yes	Yes		No
Strength/grade deck concrete and reinforcing steel	fc = 4.0 ksi Fy = 60 ksi	fc = 4.0 ksi Fy = 60 ksi	fc = 3.75 ksi Fy = 60 ksi		fc = 4.0 Fy = 60 ksi
Require or employ wearing surface	No	No	No		No
Wearing surface material and thickness used	–	–	–		–
Live load deflection requirements for bridge superstructure	L/800 L/1000	L/1500	AASHTO		AASHTO
Increased deck thickness and cover on the top mat of steel in the 1970's	Yes	Yes	No		Yes
Any other significant changes in deck thickness and/or rebar cover in last 30 years	Yes	No	Yes		Yes
Believe that excessive deck cracking and premature deterioration related to deck thickness	–	Yes	Yes		Yes
Believe that excessive deck cracking and premature deterioration related to superstructure flexibility	–	Yes	Yes		Yes
Good idea to use a standard deck thickness and vary girder spacing and reinforcing steel as necessary to standardize deck construction and in turn enhance the quality of as-built decks	Good Idea	–	Bad Idea		–

Table 3.2 Deck Thickness used by States for Different Girder Spacings (Thicknesses shown in inches)

STATE	GIRDER SPACING				
	6ft	7ft	8ft	9ft	10ft
Alabama	7	7	7	7¼	7¾
Alaska	7½	8	8¼	8½	9
Arizona	8	8	8½	9	9
Arkansas	7½	7½	7¾	8	8½
California	7¼	7⅝	8	8⅜	8⅝
Colorado	8	8	8	8	8¼
Connecticut[1]	7⅞	7⅞	7⅞	7⅞	7⅞
Delaware	8	8	8	8½	9
Florida	8	8	8	8	8
Georgia	7⅝	8	8¼	8½	8¾
Idaho	8	8	8	8⅝	8⅝
Illinois	7½	7½	7½	7½	8
Iowa	8	8	8	8	8
Kentucky	8	8	8	8	8
Louisiana[2]	8	8	8	8	8
Maine	7	7½	8¼	9	9⅝
Maryland	8½	9	9½	10	10½
Massachusetts	8	8	8⅝	9½	10½
Michigan	9	9	9	9	9
Minnesota	8⅞	8⅞	8⅞	8⅞	9
Mississippi[3]	7¾	7¾	7¾	N/A	N/A
Missouri	8½	8½	8½	8½	8½
Montana	7½	7⅞	8	8¼	8¼

[1] Minimum Values
[2] Actually 7-8", but 8" is by far the most common
[3] For prestressed concrete girders

Table 3.2 Deck Thickness used by States for Different Girder Spacings (Thicknesses shown in inches)

STATE	GIRDER SPACING				
	6ft	7ft	8ft	9ft	10ft
Nebraska	7½	7½	7½	7½	8
Nevada	8	8	8	8	8
New Hampshire	8	8	8	8 (or as required)	
New Mexico	7¾	8¼	8¾	9¼	10
New York	9½	9½	9½	9½	9½
New York Thruway Authority	9½	9½	9½	9½	9½
North Carolina	8	8	8½	8¾	9
North Dakota	8	8	8	8	8
Ohio	8½	8½	8½	8¾	9
Oklahoma	8	8	8	8	8
Oregon	7¾	7¾	7¾	7¾	8
Pennsylvania	7⅞	7⅞	7⅞	8¼	8⅝
Puerto Rico	8	8	8	N/A	N/A
Rhode Island	7½	7½	7½	7½	7½
South Carolina	8	8	8½	9	9½
South Dakota	8	8	8	8¼	8⅝
Tennessee	8	8	8	8½	8¾
Texas	8	8	8	8	N/A
Virginia[4]	7½	8	8½	8½	8½
Wisconsin	8	8	8½	9	9½
Wyoming	8	8	8	8	8¼

[4] For Steel Girders

Table 3.3 Summary of Southern State Bridge Deck Design Parameter Values

STATE	USE DEICING SALT ON DECK	REQUIRE OR EMPLOY WEARING SURFACE	DECK REBAR COVER		DECK CONCRETE STRENGTH f'_c ksi	DECK THICKNESS (in) FOR GIRDER SPACING OF				
			TOP (in)	BOTT. (in)		6'	7'	8'	9'	10'
ALABAMA	NO	NO	2	1	4.0	7	7	7	7¼	7¾
ARKANSAS	YES	NO	2	1	4.0	7½	7½	7¾	8	8½
FLORIDA	NO	YES	2	2	4.5	8	8	8	8	8
GEORGIA	YES	NO	2¼ or 2¾	1	3.0-3.5	7⅝	8	8¼	8½	8¾
LOUISIANA[1]	YES	NO	2	1	3.2	8	8	8	8	8
MISSISSIPPI[2]	YES	NO	2¼	1	4.0	7¾	7¾	7¾	NA	NA
NORTH CAROLINA	YES	YES	2½	1¼	3.5	8	8	8½	8¾	9
SOUTH CAROLINA	YES	NO	2½	1	4.0	8	8	8½	9	9½
TENNESSEE	YES	NO	2½	1	3.0	8	8	8	8½	8¾
TEXAS	YES	NO	2	1½	4.0	8	8	8	8	NA

[1] Typical Deck Thickness Values Shown.
[2] For Prestressed Concrete Girders

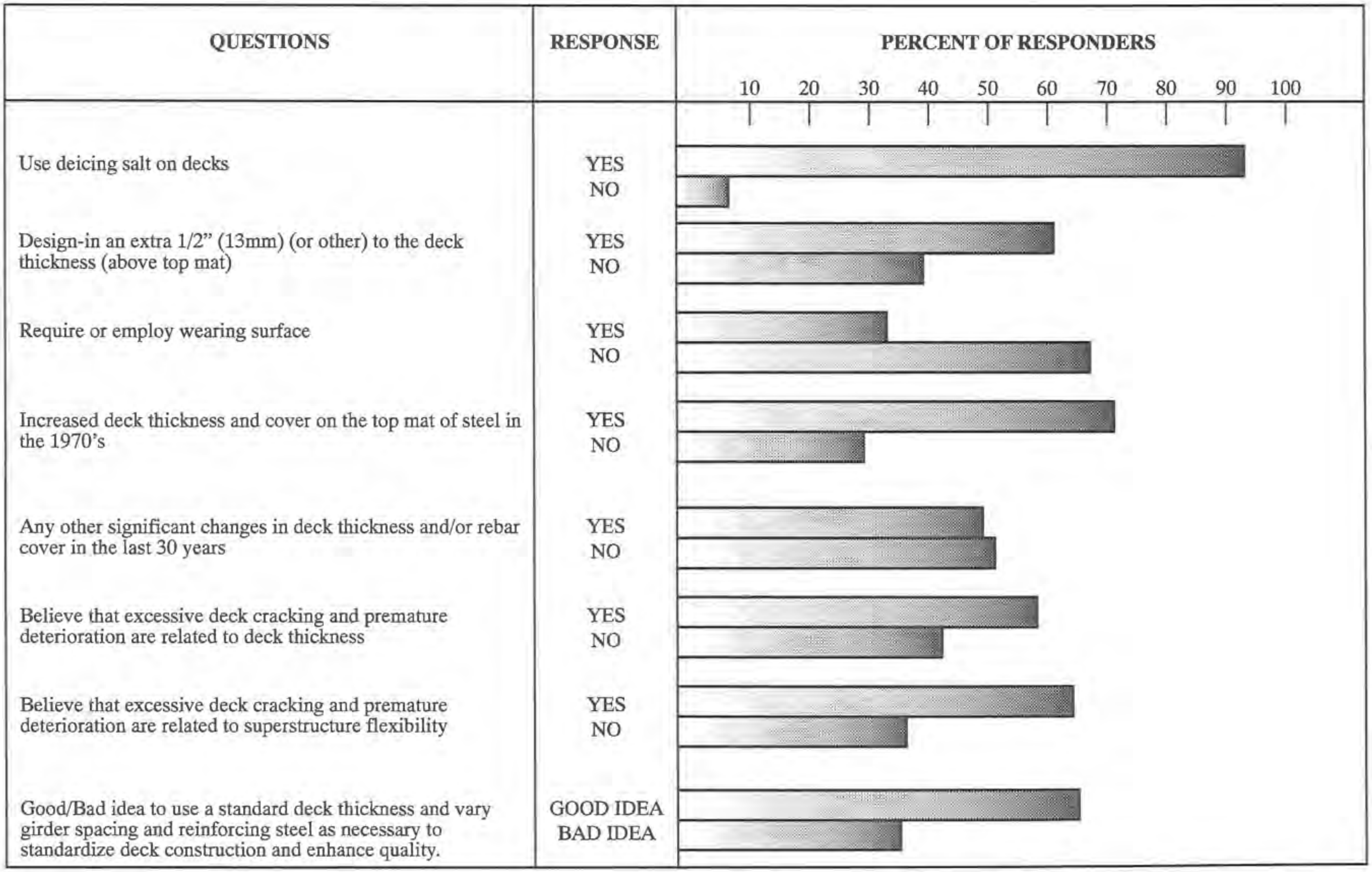


Figure 3.1 Summary of State DOT Responses to Primary Questions on Survey Questionnaire

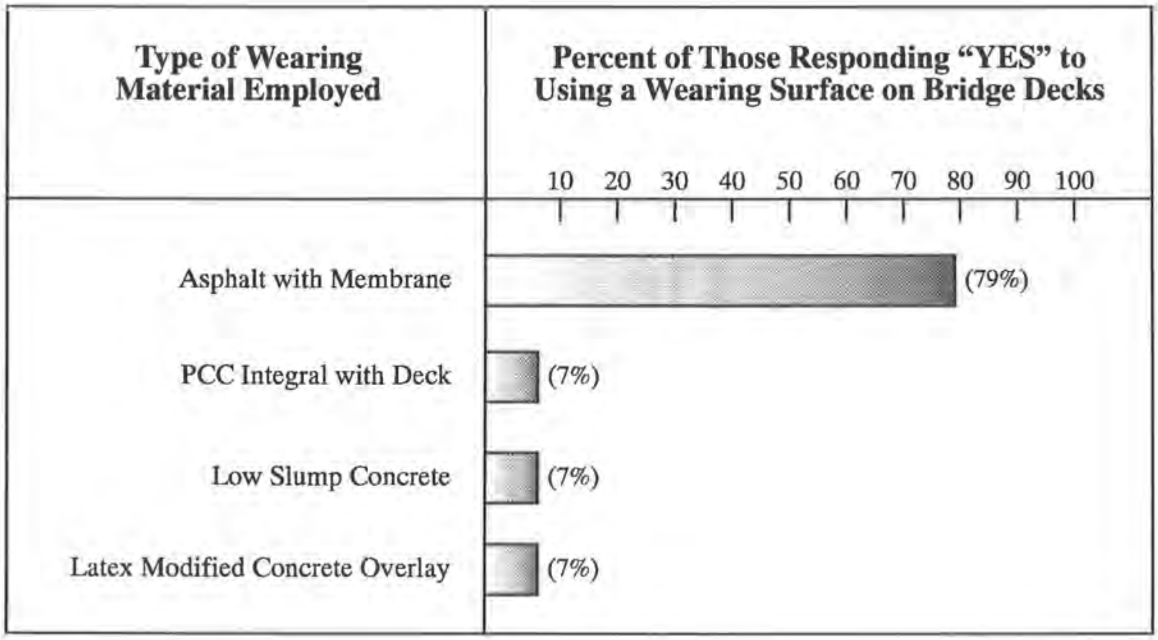
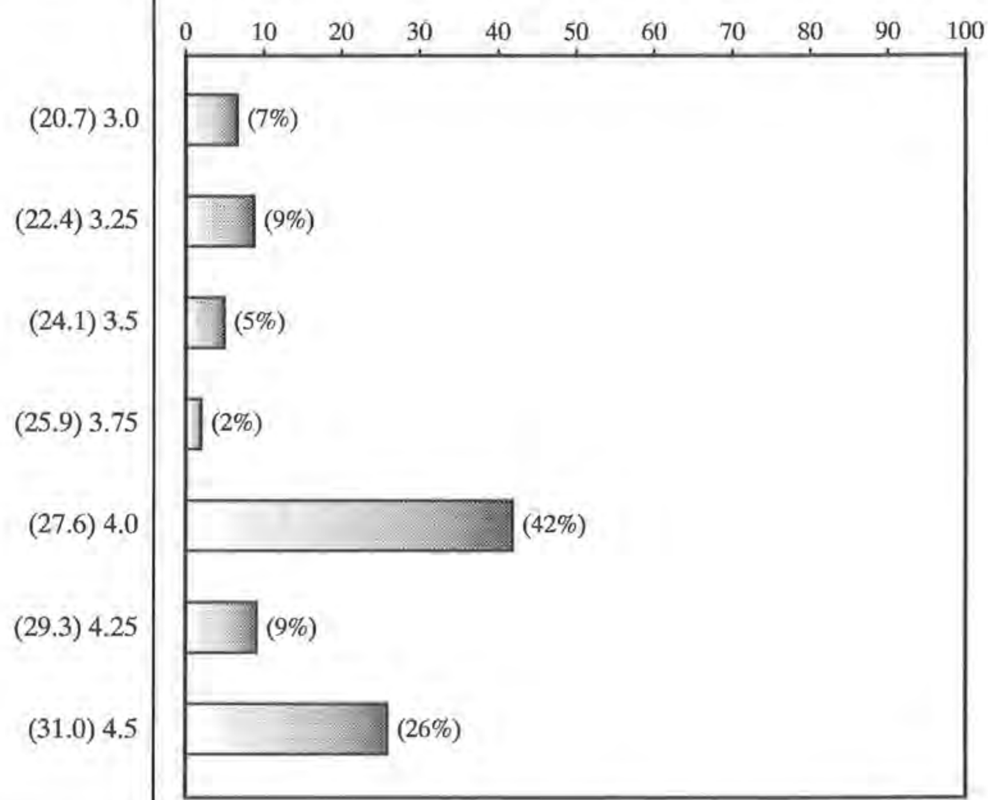


Figure 3.2 Summary of Wearing Materials Employed

DECK CONCRETE STRENGTH, f'_c, ksi (MPa)	PERCENT OF RESPONDERS*



*Five States Quoted a Small Range for f'_c. Midpoint values were used in those instances

Figure 3.3 Summary of Deck Concrete Strength (f'_c) Used by Responders (All use F_y=60 ksi (413.7 MPa) for Rebar)

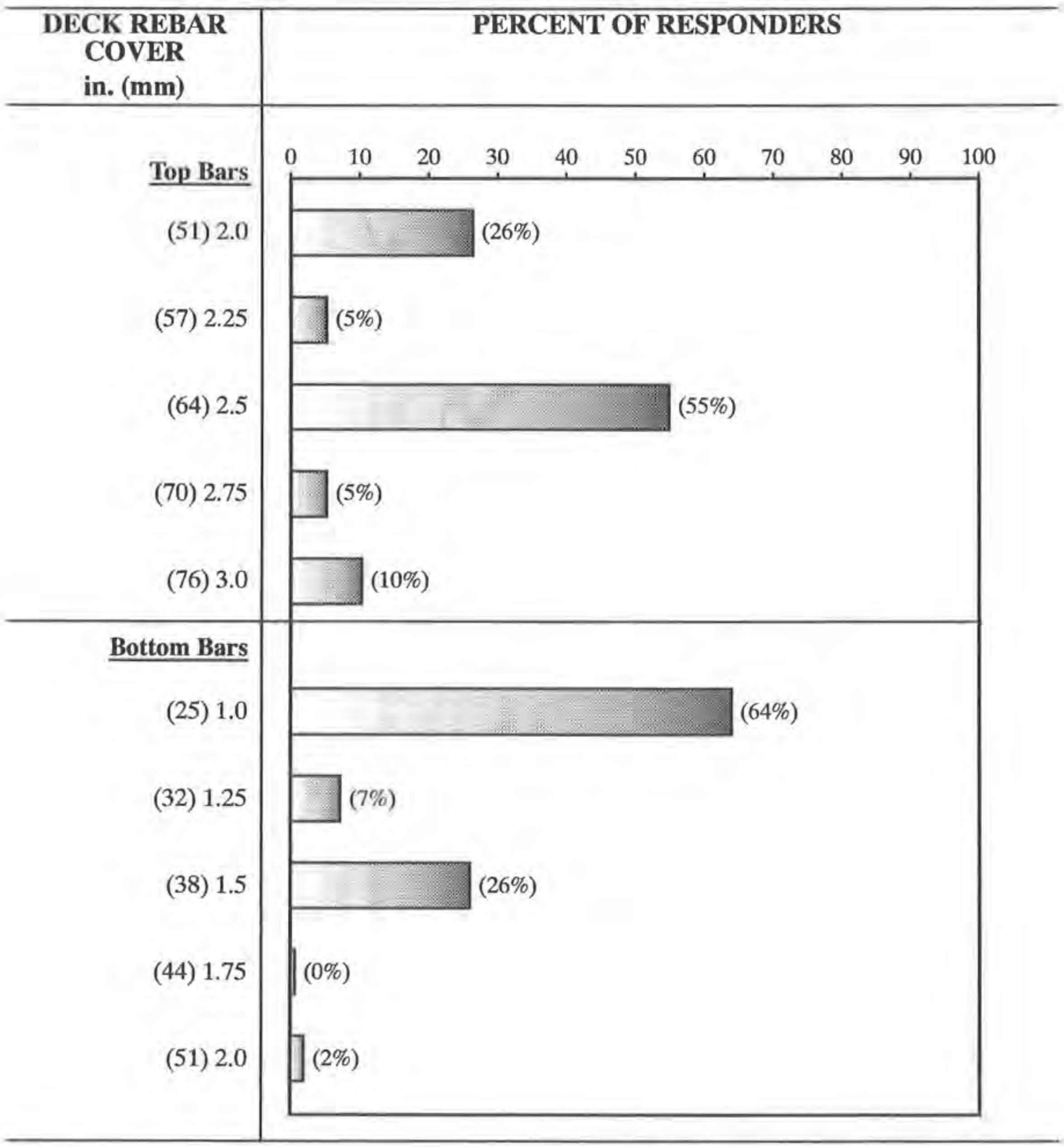


Figure 3.4 Summary of Deck Top and Bottom Covers

4. THEORETICAL CONSIDERATIONS

4.1 General

Theoretical considerations of bridge deck and superstructure states of loading, behavior, and performance are considered here in an effort to identify the parameters affected by the deck thickness and in what manner. The deck parameters and behaviors are examined using various theoretical approaches to help assure that all relevant parameters are identified. In a later chapter, the most important of the parameters will be determined and examined via parameter sensitivity study. The theoretical considerations discussed below are not presented in any particular order, but rather to simply getting all of the participating parameters "out on the table."

4.2 Behavior of Beam-and-Slab Decks

A typical steel composite beam-and-slab bridge is shown in Fig. 4.1. Beam-and-slab decks are typically categorized into those with the beams at close centers or touching which are referred to as "contiguous beam-and-slab" as shown in Figs. 4.2 and 4.3, and those with beams at wide centers are referred to as "spaced beam-and-slab" as shown in Figs. 4.1, 4.4, and 4.5 (16). Spaced beam-and-slab decks commonly have the beams at about 6 to 12 ft. centers. All, or nearly all, of the ALDOT beam-and-slab deck bridges fall into his latter category. Additionally, most of their bridges are beam-and-slab deck bridges, as are most of the highway bridges in the United States.

When a load is place above one beam of a spaced beam-and slab deck, the slab does not necessarily deflect transversely in a single wave as shown in Fig. 4.3b, but sometimes in a series of waves between beams. This is particularly the case if the beams have high torsional stiffnesses, as do box beams, when the beams may twist little so that the slab deflects in a series of transverse steps as shown in Fig. 4.5. (16).

Fig. 4.1 Steel Composite Beam-and-Slab Bridge at Newburgh, Scotland (16)

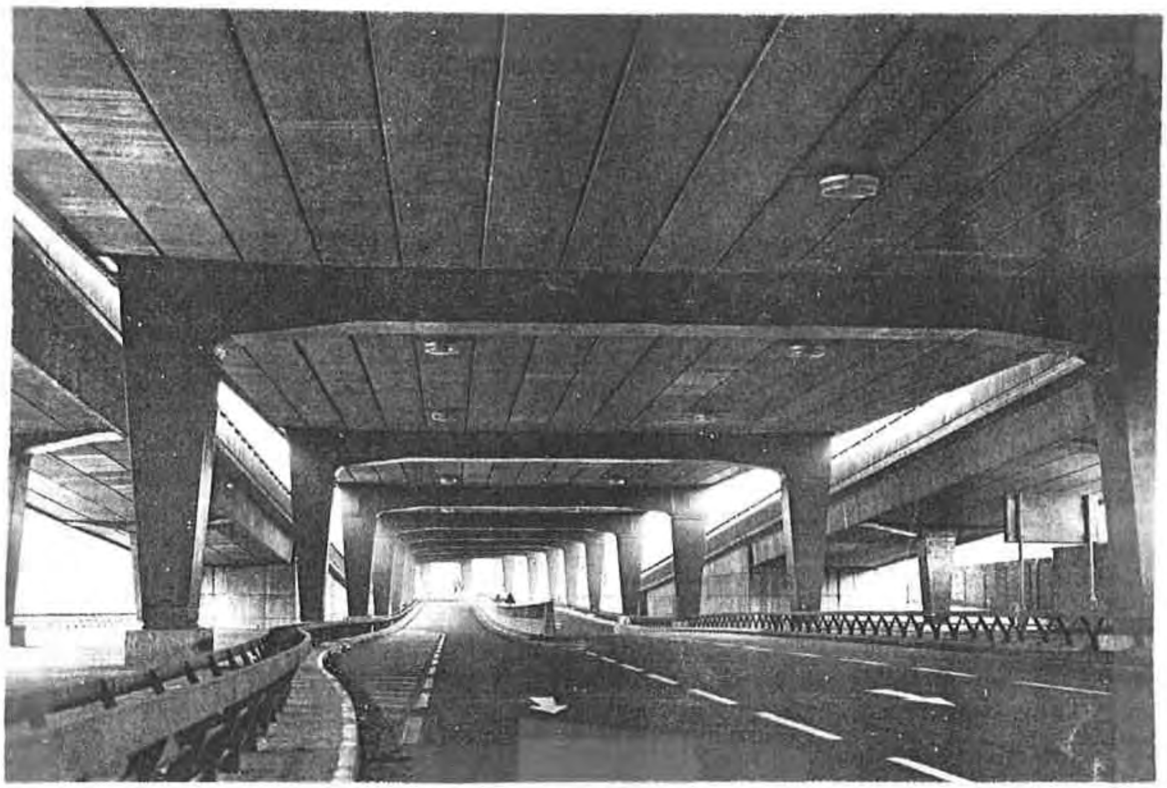
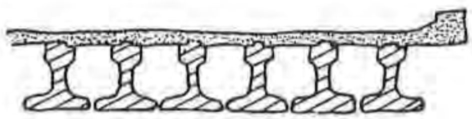


Fig. 4.2 Beam-and-Slab Deck with Contiguous Concrete Beams (16)



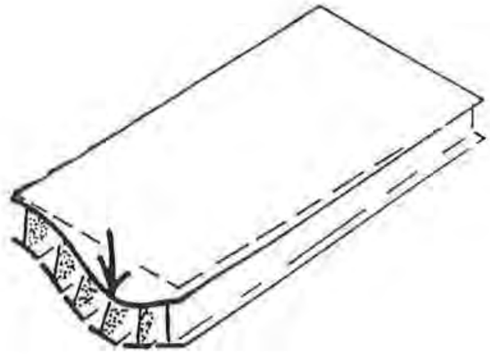


Fig. 4.3(a). Contiguous Beam-and Slab Deck (16)

4.3(b). Slab of Contiguous or Spaced Beam-and-Slab Deck Deflecting in Smooth Wave (16)

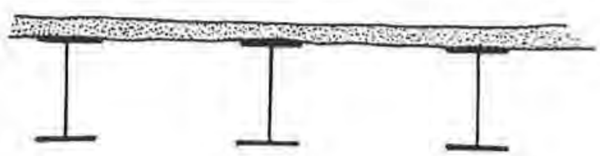


Fig.4. 4 Spaced Beam-and-Slab Decks with Steel I-beams and Concrete Slab (16)

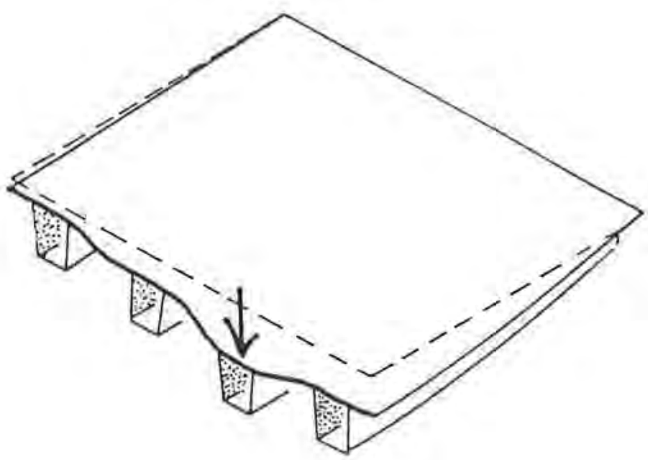


Fig. 4.5 Spaced Beam-and-Slab Deck Deflecting in a Series of Steps or Waves (16)

It should be noted that nearly all of the I65 and I20/59 system bridges through downtown Birmingham are of the type shown in Figs. 4.1 and 4.4 and have low torsional stiffnesses. Thus their transverse deflections under truck wheel loads would be similar to Fig. 4.3b except at locations of transverse diaphragms where deflections would be more like Fig. 4.5.

A bridge slab deck behaves much like a flat plate which is structurally continuous for the transfer of moments and torsions in all directions within the plane of the plate. When a concentrated load is placed on part of a slab, the slab deflects locally in a "dish" causing a two-dimensional system of moments and torsions which transfer and share the load to neighboring parts of the deck which are less severely loaded, as shown in Fig. 4.6.

As indicated in Fig. 4.6, moments in the transverse and longitudinal directions associated with the 'dish' deflections and curvatures, would tend to cause tensile stresses on the bottom of the deck and could cause tension cracking of the type observed on the bottom side of the Birmingham bridge decks (see Fig. 4.7). This crack pattern can also be seen in Figs. 4.8 and 4.9 from laboratory experimental model slab testing..

To gain a quantitative insight as to the possibility of truck wheel loads causing structural cracking on the underside of the deck as shown in Fig. 4.7, one can assume the deck as behaving as a unit width beam spanning in the transverse direction and supported on discreet elastic foundation elements (the girders) as shown in Fig. 4.10.

Assuming a simple span of 70 ft with the P load at the centerline,

$$K_G = \frac{48EI}{L^3} = \frac{48(29{,}000^K/in^2)\ (9040\ in^4)}{70^3\ x\ 1728\ in^3}$$

$$K_G = 21.23^K/in$$

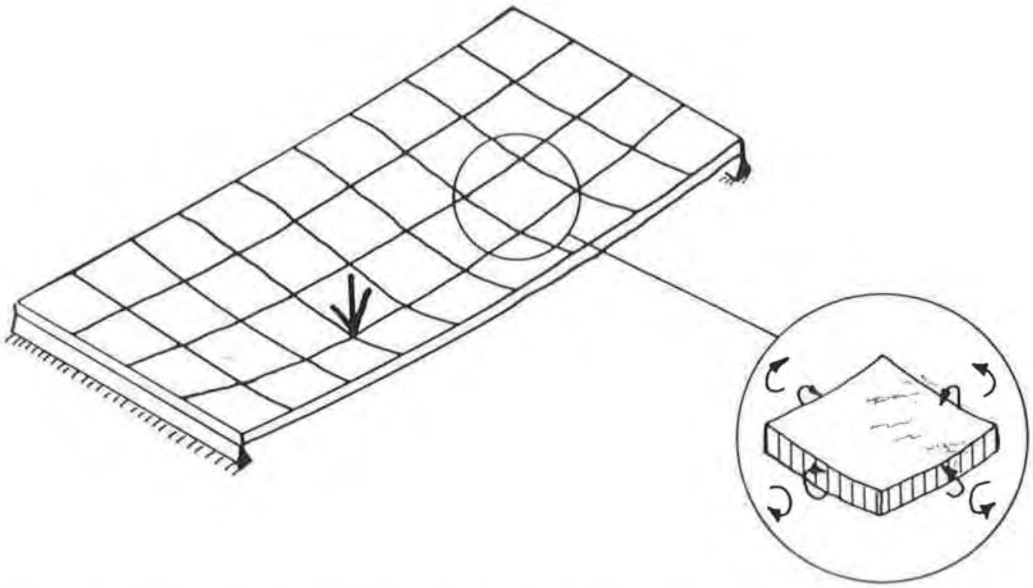


Fig. 4.6 Load Distribution in Bridge Slab Deck by Bending and Torsion in Two Directions (16)

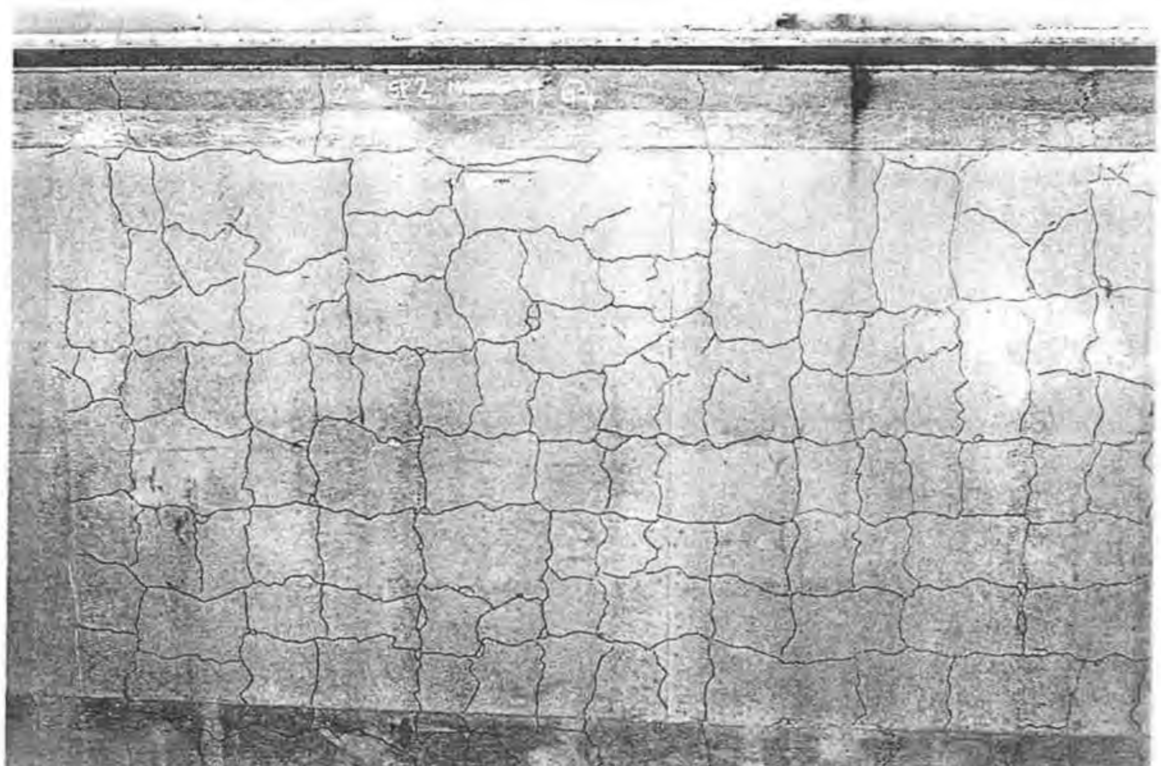


Fig. 4.7 Close-up of Underside of I-65 Bridge at Midspan with Hairline Cracks Highlighted (24)

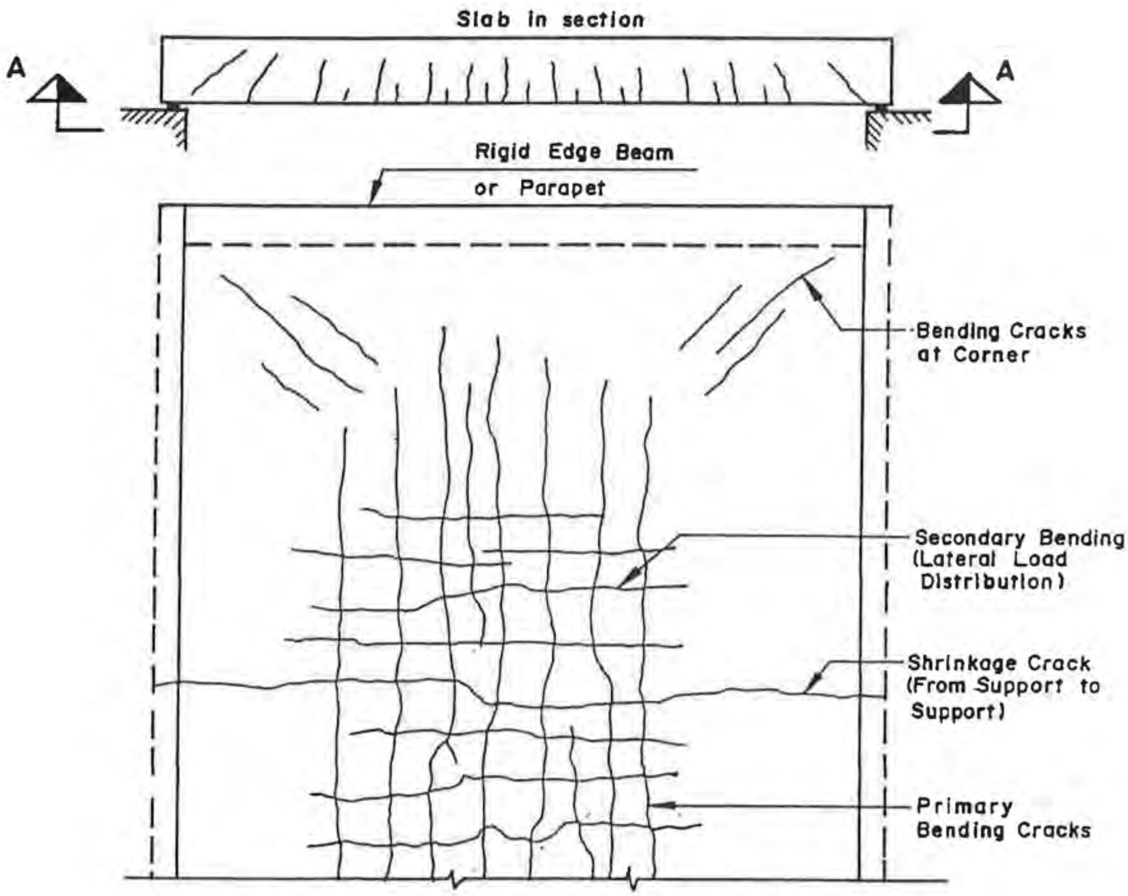


Fig. 4.8 Underside of Slab Bridge Deck (Ref. Unknown)

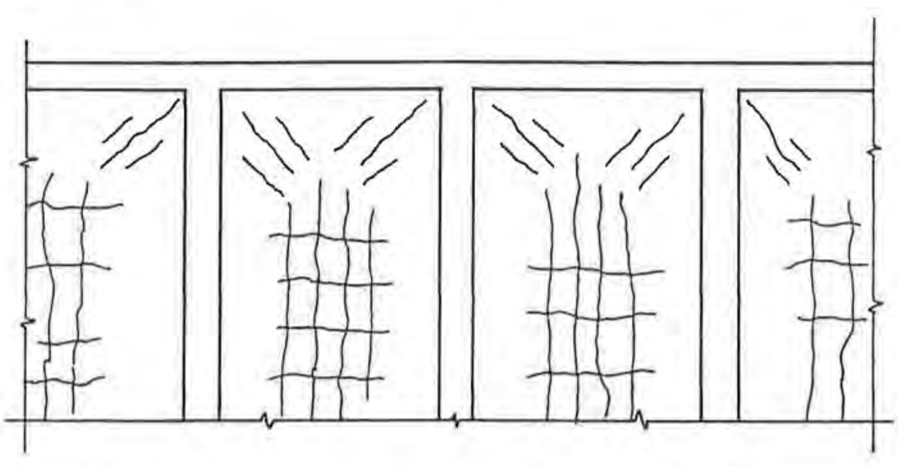


Fig. 4.9 Underside of Slab-Girder Bridge Deck (Ref. Unknown)

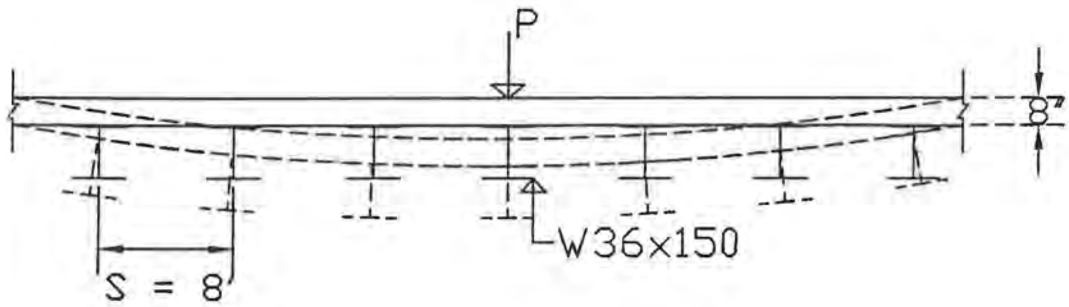


Fig. 4.10. Modeling of Deck as a Beam on an Elastic Foundation

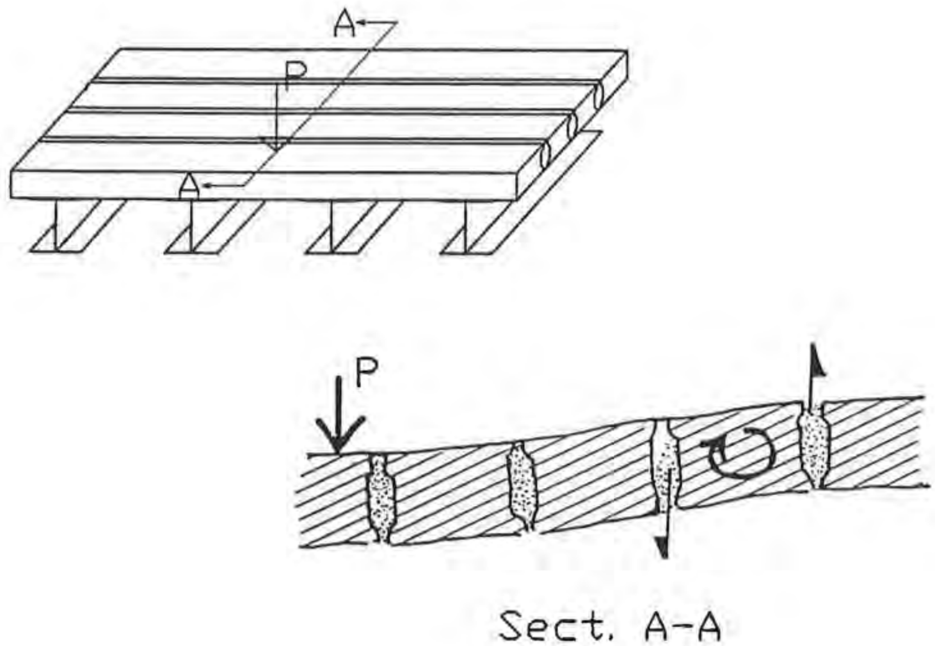


Fig. 4.11 Differential Deflections of Beams in Shear-Key Deck Resisted by Flexure and Torsion of Beams

Smearing the K_G over the 8 foot spacing of the girders (as per beam on elastic foundation theory) yields,

$$k = \frac{K_G}{S} = \frac{21.23^K/in}{96\ in} = 0.2212^K/in^2$$

$$\beta = \sqrt[4]{\frac{k}{4\ EI_{BEAM}}}$$

where $$E = E_c = 57{,}000\ \sqrt{4000} = 3{,}605\ ksi$$

$$I_{BEAM} = \frac{12(8)^3}{12} = 512\ in^4$$

$$\beta = \sqrt[4]{\frac{0.2212^K/in^2}{4\ x\ 3{,}605^K/in^2\ x\ 512\ in^4}}$$

$\beta = 0.01316/\text{in}$

Checking spacing of the discreet springs yields,

$$96\ in = S\ v.s.\ \frac{\pi}{4\beta} = \frac{\pi}{4\ x\ 0.01316/in} = 60\ in$$

Therefore, spacing of discreet springs (girders) is somewhat excessive for accurate modeling, however one can still proceed to get a reasonable approximation of deck moments under the P-load.

Checking treating the deck beam as an infinite length beam yields,

$$L_{TRANSVERSE} = L\ vs\ \frac{3\pi}{4\beta} = 180in = 15ft$$

Therefore it can be treated as an infinite beam.

Working with just one concentrated load, i.e., one wheel load, yields

$$M_{MAX} = \frac{P}{4\beta} (C_{\beta X})$$

$$= \frac{16^{K}}{4 \ x \ 0.01316/in} (1.0)$$

$$= 304^{\text{in-K}}$$

$$\sigma_{MAX} = \frac{M_{\max}}{S} = \frac{304^{in-K}}{\frac{12(8)^2}{6} \ in^3} = 2.37 \ ksi$$

Obviously this stress level would crack the concrete on the underside (longitudinal crack).

In the longitudinal direction, a unit width strip of the deck can also be modeled as abeam on an elastic foundation similar to the prefabricated shear-key beam elements shown in Fig. 4.11. Maximum moments under the P-load would be of the same order of magnitude as those in the transverse direction, as would the maximum bending stresses. Again, this stress level would crack the underside of the deck, except in the transverse direction. Thus, moments in the transverse and longitudinal directions under a truck wheel load will result in deck structural cracking on the underside in the manner displayed in the photo of Fig. 4.7.

The principal function of the slab in beam-and-slab bridges is to provide the roadway surface and to transmit the applied loads to the beams/girders. In turn, the girders transmit the load to the abutment and/or intermediate bents, which take the loads to the foundation. The load path described above is illustrated in Fig. 4.12(b) by the squiggly lines. The load causes the beam-and-slab system to displace as shown in Fig. 4.12(c). If linear behavior is assumed, the load to each girder is proportional to its displacement. As expected, the girder near the location of the load application carries more load than those away from the applied load. Equilibrium requires that the

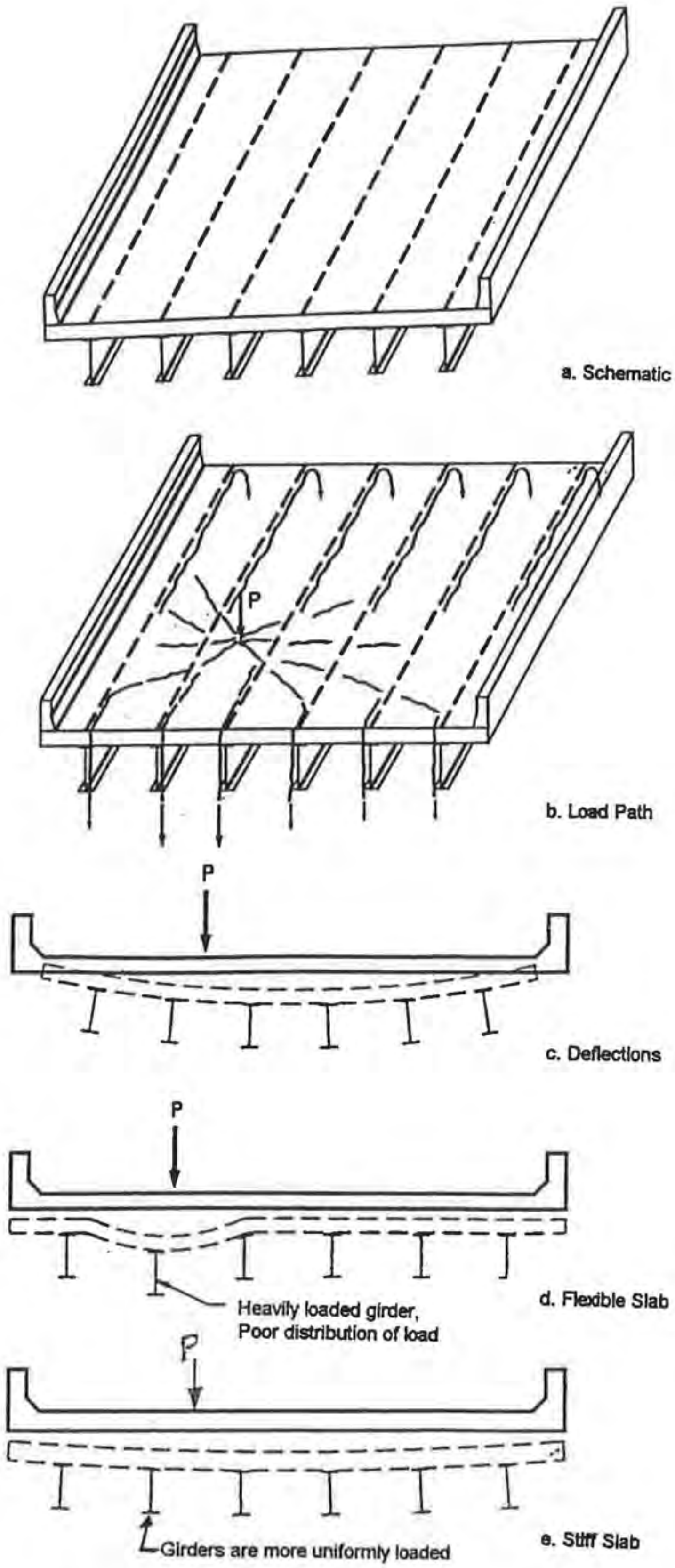


Fig. 4. 12 Beam-and-Slab Bridge (6)

summation of the load carried by all the girders equals the total applied load. The load carried by each girder is a function of the relative stiffness of the components that comprise the beam-and-slab system. The two principal components are the slab and the girders.

The effect of relative stiffness is illustrated by considering the two beam systems shown if Figures 4.12(d) and 4.12(e). The system shown in Figure 4.12(d) has a slab that is relatively flexible compared to the girder. Note the largest deflection is in the girder under the load and the other girder deflections are relatively small. Now consider the system shown in Figure 4.12(e) where the slab is stiffer than the previous case. Note the load (deflection) is distributed to the girders more evenly, therefore the maximum load to the most severely loaded girder is less than shown in Figure 4.12(d).

From observing the deflections in Figures 4.12 c, d, e, obviously the transverse bending stiffness of the slab is important in distributing an applied load. Not so obvious is that the torsional stiffness of the slab (since it must twist in the longitudinal direction to accommodate the "local" transverse deflections shown in Fig. 4.12) is also important. Both the bending and torsional stiffness of the slab vary as the cube of the slab depth. Thus, the relative bending and torsional stiffnesses of slabs of depth 6 to 9 inches would be as indicated in Table 4.1. Obviously, increasing the slab thickness a small value greatly enhances its bending and torsional stiffnesses. Note in Table 4.1 that increasing the deck thickness from 7" to 8" would increase both the deck bending and torsional stiffnesses by 50%. The more uniform distribution of load to the girders provided by the stiffer slab would reduce peak live load stresses in the girders, and in-turn enhance their fatigue/service life.

Table 4.1 Relative Bending and Torsional Stiffnesses for Slabs 6 to 9 Inches Thick

D (in)	D^3 (in^3)	$D^3/D^3_{6"}$
6	216	1.00
7	343	1.59
8	512	2.37
9	729	3.38

4.3 Influence Lines for Beam-and-Slab Load Distribution Coefficients

Hambly (16) presents charts for the determination of influence lines for various points across the cross-section of simply supported right decks of slab, beam-and-slab and cellular construction. The charts are designed to help the bridge engineer make initial choices of type of superstructure and of deck dimensions via allowing rapid determinations of load distribution and deflection characteristics, and maximum design moments. Since we are only interested in beam-and-slab bridges, only the charts and parameters applicable to this bridge type will be discussed. Figures 4.13a,b,c, are Hambly's charts after modification to show only the portions applicable to beam-and-slab bridges.

The first step in the analysis of any deck with these charts is to notionally subdivide the deck unto a number of parallel "beams" as shown in Fig. 4.14. The physical characteristics of the deck for beam-and-slab systems can then be summarized by two nondimensional parameters which relate the various stiffnesses of the structure. The parameters are as follows.

> f, the flexural stiffness ratio. This relates the transverse flexural stiffness of the slabs between 'beams' to the longitudinal flexural stiffness of the 'beams', ie,

$$f = \frac{\textit{transverse flexural stiffness of slab between beams}}{\textit{longitudinal flexural stiffness of beam}} \tag{4.1}$$

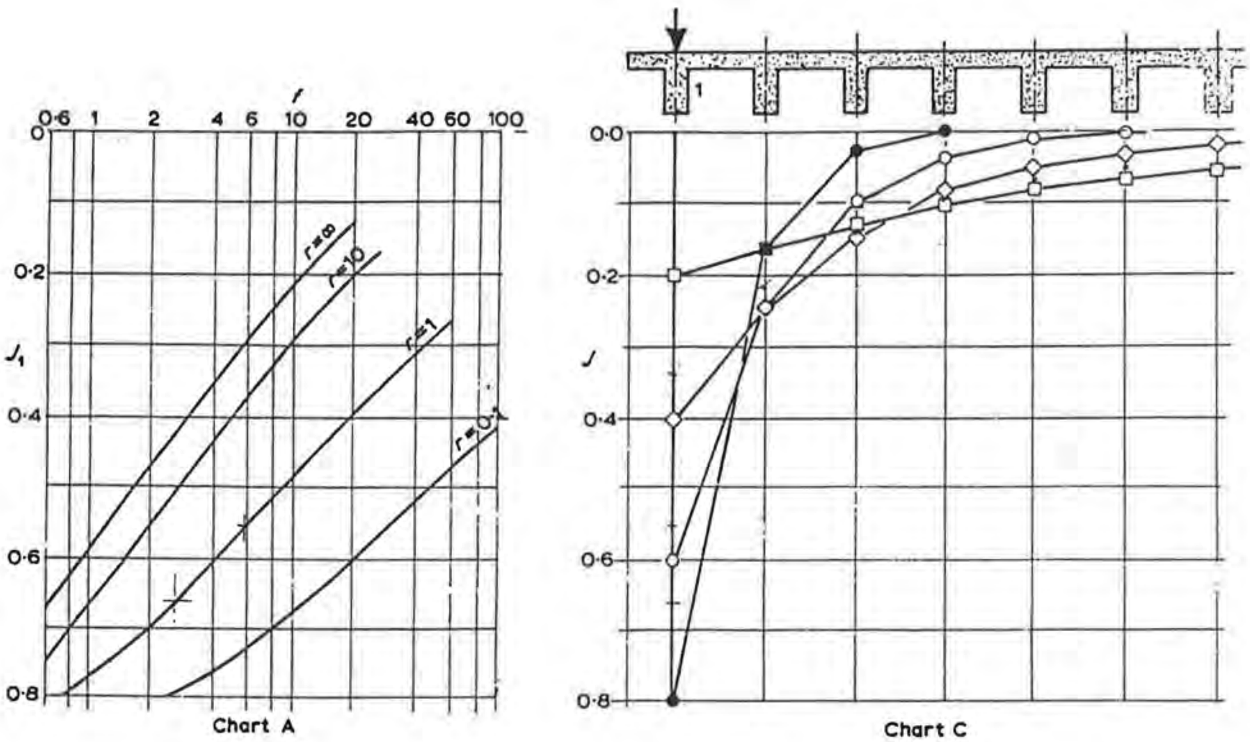


Fig. 4.13a. Influence Lines for Edge Beam (modified from Ref. 16)

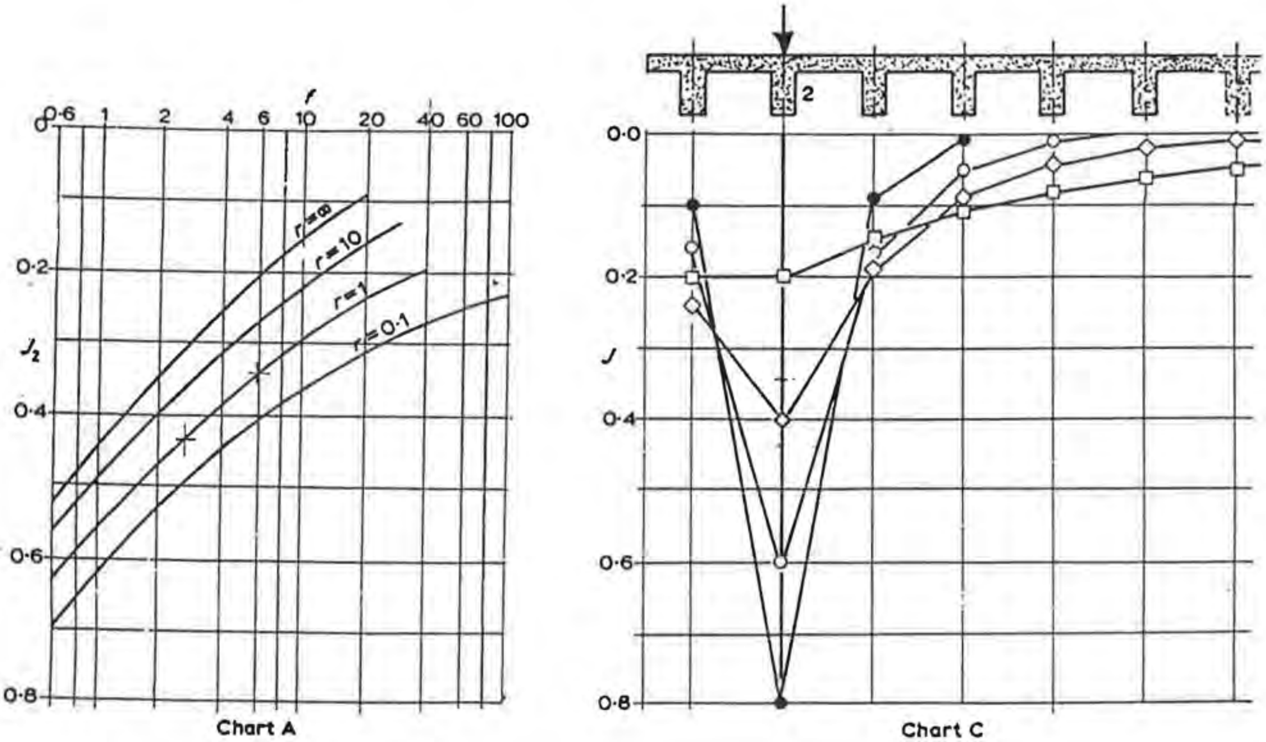


Fig. 4.13b. Influence Lines for Beam Next to Edge (modified from Ref. 16)

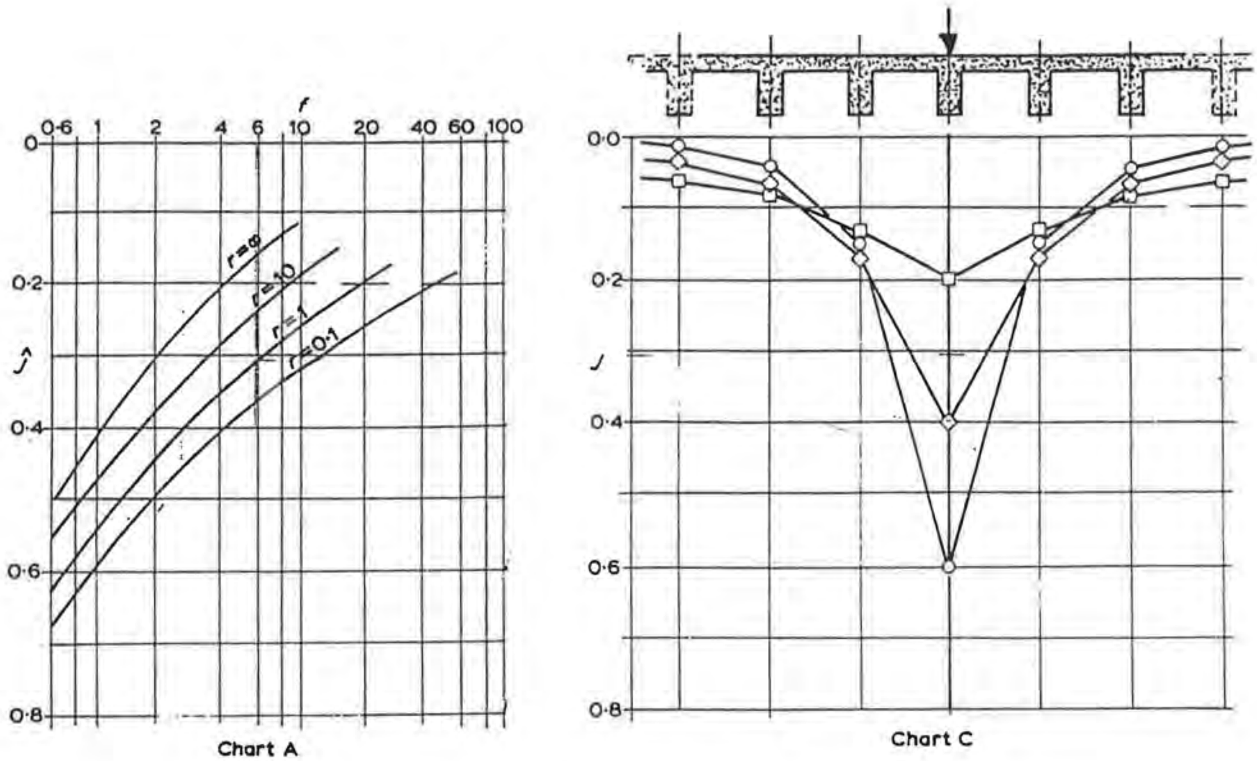


Fig. 4.13c Influence Lines for Beam Far From Edge (modified from Ref. 16)

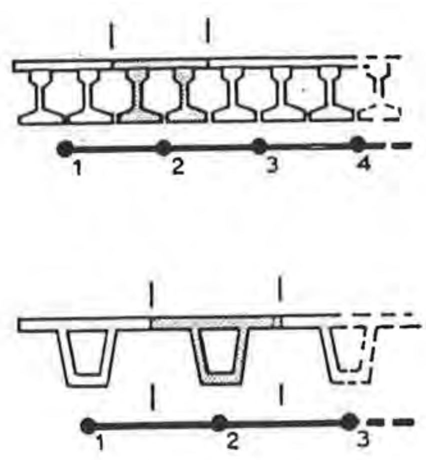


Fig. 4.14 Deck Cross-Section Divided into 'Beams' (16)

r, the rotational stiffness ratio. This relates the torsional stiffness of the slab and beams to the transverse flexural stiffness of the slab, ie.,

$$r = \frac{\textit{torsional stiffness of slab and beams}}{\textit{traverse flexural stiffness of slab}} \tag{4.2}$$

For beam-and-slab decks the nondimensional parameters are

$$f = 0.12 \frac{i}{l^3} x \frac{L^4}{I} \tag{4.3}$$

$$r = 5 \frac{G}{E} x \frac{l}{i} x \frac{C}{L^2} \tag{4.4}$$

where

L = span
l = 'beam' spacing
i = transverse moment of inertia of slab per unit length
I = moment of inertia of 'beam'
C = torsion constant of 'beam' + l x (transverse torsion constant of slab per unit length).

If the slab has depth d and v = 0.2 so that G/E = 0.42, Equations (4.3) and (4.4) reduce to,

$$f = 0.01 \frac{d^3}{l^3} x \frac{L^4}{I} \tag{4.5}$$

$$r = 25 \frac{l}{d^3} x \frac{C}{L^2} \tag{4.6}$$

Paraphrasing from Hambly, Chart A in Figures 4.13a, b, c enables the user to derive the peak value of the influence line for a particular 'beam' from the nondimensional parameters f and r. Chart C is used to derive the rest of the influence line through the peak value. The influence value J at any point across an influence line for a ' beam' is the fraction of the total moment on the deck, due to a load above the point, that is carried by the 'beam'. Alternatively it can be thought of as the moment (or deflection) in the 'beam' expressed as a fraction of the total moment (or deflection) that the 'beam' would experience if it carried the load by itself without load distribution to the rest of the deck. The Charts C have been called 'influence lines' as it is generally more convenient to use them as such. More strictly they are distributions of moment or deflection for the first harmonic of a line load on the relevant 'beam'. As long as the 'beams' are not very close together (so that distribution of harmonics higher than the first is not significant), the reciprocal theorem can be applied with little error to moments in the 'beams' in addition to deflections as is strictly correct. Consequently, Charts C can be used either as distributions of moment and deflection for a load on a particular 'beam' or as influence lines.

Charts C only give the values of each influence line at the 'beam' positions. The shape of the line between 'beams' depends on the rotational stiffness parameter r. If r is small as it is for the decks of Fig. 4.15 (a) and (b), the deck cross-section distorts in a smooth curve. In contrast, if r is large, as for the decks of Fig. 4.15 (c) and (d), the distortion and influence line are 'stepped'.

Application Example - Spaced Box Beam-and-Slab. Figure 4.16 shows a beam-and-slab deck constructed of prestressed box beams supporting a reinforced concrete slab. The deck is supporting an abnormal heavy vehicle of four axles, each of four wheels, located at midspan between the edge beam and the next beam to the edge. The reinforced concrete has a modular ratio m = 0.8 relative to the prestressed concrete.

For analysis, the deck is divided into four identical 'beams' at l = 3.2 m spacing. For each 'beam'

$$I = 0.30 \text{ m}^4$$

$$C = 0.26 + 3.2 \left(\frac{0.8 \ x \ 0.2^3}{6} \right) = 0.26 m^4$$

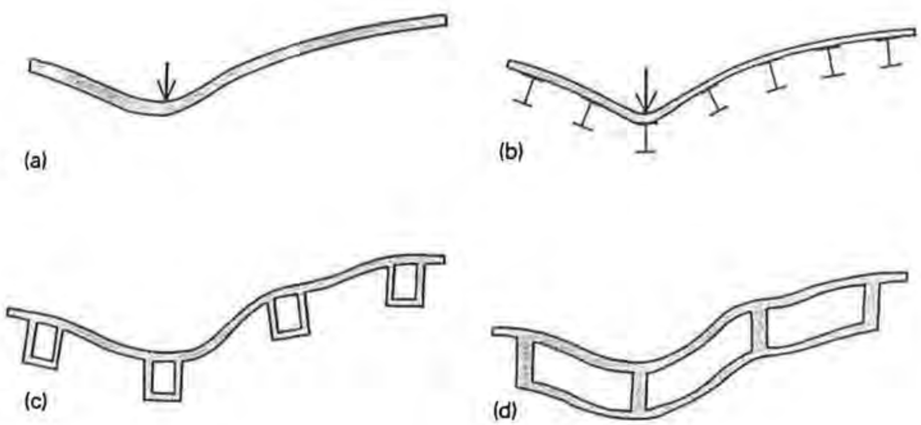


Fig. 4.15 Cross-Section of Various Decks: (a) and (b) Distorting in Smooth Curve
(c) and (d) Distorting in Series of Steps (16)

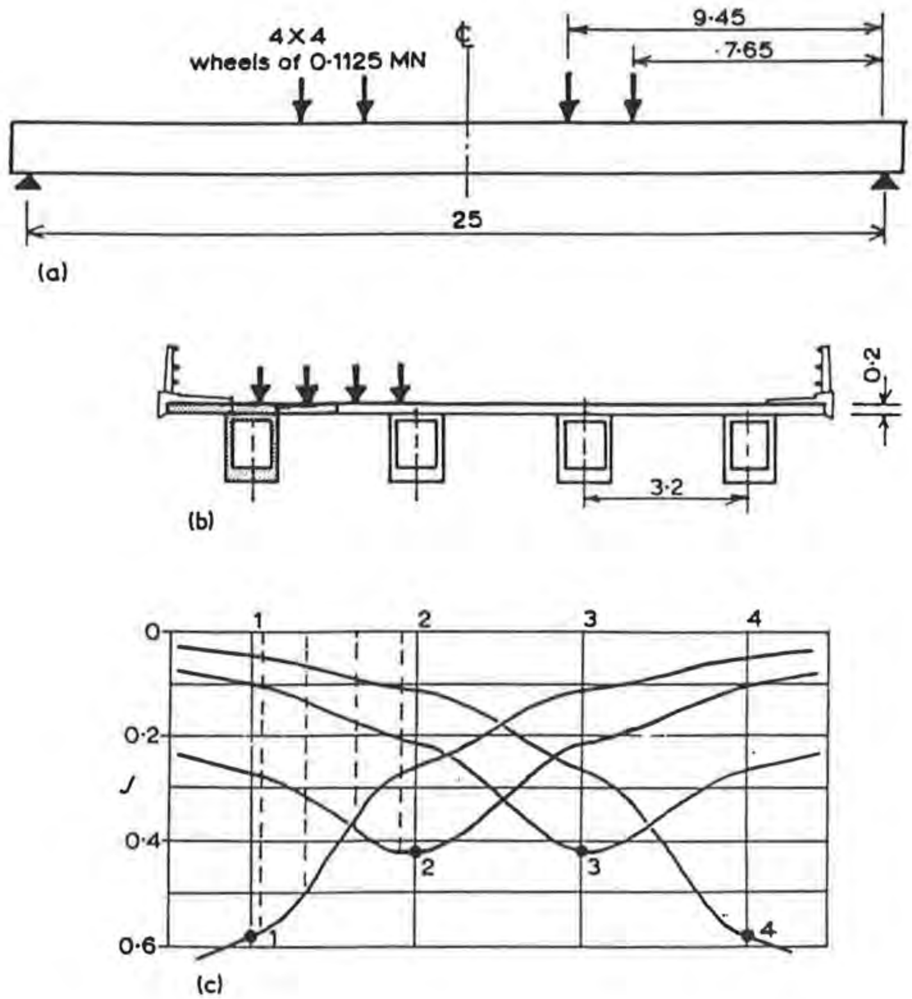


Fig. 4.16 Influence Lines for Beam-and-Slab Deck: (a) Elevation; (b) Cross-Section; (c) Influence Lines for 'Beams' (16)

With L = 25.0 we obtain from Equations (4.5) and (4.6)

$$f = 0.01 \; x \; 0.8 \; x \; \frac{0.2^3}{3.2^3} \; x \; \frac{25^4}{0.30} = 2.5$$

$$r = 25 \; x \; \frac{3.2}{0.8 \; x \; 0.2^3} \; x \; \frac{0.26}{25^2} = 5.2$$

Using the charts of Figures 4.13a, b, c, we find that for the above values of f and r

$J_1 = 0.55$	for edge 'beam' influence line
$J_2 = 0.39$	for 'beam' next to edge influence line
$\hat{J} = 0.37$	for internal 'beam' influence line.

Also using Charts C of Figures 4.13a, b, c, we obtain influence lines for edge and next to edge (and internal 'beams' not here relevant)

$$\begin{array}{llllll} & \downarrow & & & & \\ J & 0.55 & 0.25 & 0.11 & 0.05 & \Sigma = 0.96 \\ & & \downarrow & & & \\ J & 0.25 & 0.39 & 0.19 & 0.09 & \Sigma = 0.92 \\ & & & \downarrow & & \\ (J & 0.08 & 0.15 & 0.37 & 0.17...) & \end{array}$$

The sums of the influence values above are close to unity because the load distribution characteristics of the deck are not very good, and the deck is nearly wide enough for the moments in 'beams' 1 and 2 to be little influenced by loads or additional beams on the far side. On factoring up the J values above to give sums of unity we obtain

$$\begin{array}{llllll} & \downarrow & & & & \\ J & 0.58 & 0.26 & 0.11 & 0.05 & \sum = 1.0 \\ & & \downarrow & & & \\ J & 0.27 & 0.42 & 0.21 & 0.10 & \sum = 1.0 \end{array}$$

These influence lines are plotted in Fig. 4.16 (c) together with the mirror image lines appropriate to 'beams' 3 and 4.

For the abnormal heavy vehicle loading, the midspan moment per longitudinal line of wheel is

$$M = 0.1125(7.65 + 9.45) = 1.924 \text{ MNm}$$

Since the lines of wheels coincide with values on influence lines for 'beams' 1, 2, 3, and 4 of

'beam' 1	0.57	0.49	0.35	0.27
'beam' 2	0.28	0.31	0.38	0.42
'beam' 3	0.11	0.13	0.17	0.20
'beam' 4	0.05	0.07	0.09	0.11

the total moments in the 'beams' are

$$M^1 = 1.924(0.57 + 0.49 + 0.35 + 0.27) = 3.23 \text{ MNm}$$

$$M^2 = 1.924(0.28 + 0.31 + 0.38 + 0.42) = 2.67 \text{ MNm}$$

$$M^3 = 1.924(0.11 + 0.13 + 0.17 + 0.20) = 1.17 \text{ MNm}$$

$$M^4 = 1.924(0.05 + 0.07 + 0.09 + 0.11) = 0.62 \text{ MNm.}$$

In summary, from Charts A and C in Figures 4.13a, b, c for beam-and-slab systems, along with the associated definitions and equations of the chart parameters f and r, the following observations are made.

1. From Charts C, the smaller the J value of the beam/girder where the load is applied the better the load distribution characteristics of the deck.

2. A more uniform distribution of loading to the girders would reduce the girder maximum bending moments and stresses. In-turn, this would lead to greater girder fatigue/service life. Also a more uniform distribution of loading would reduce girder deflections and relative deflections, which should reduce deck bending moments and stresses, and in-turn reduce deck structural and fatigue cracking and enhance the deck's fatigue/service life.

3. For a given f, the larger the rotational stiffness ratio parameter, r, the smaller the J value at the location of the load, and the better the load distribution characteristics of the deck.

4. For a given r, the larger the flexural stiffness ratio parameter, f, the smaller the J value at the location of the load, and the better the load distribution characteristics of the deck.

5. The larger the stiffness ratio parameters f and r, the smaller the J value at the location of the load, and the better the load distribution characteristics of the deck.

6. From the definition of the f parameter associated with Charts A and C, ie.,

$$f = \frac{\textit{transverse flexural stiffness of slabs between beams}}{\textit{longitudinal flexural stiffness of the beams}}$$

and from (5) above, increasing deck thickness (other parameters held constant) would result in a far greater increase in transverse flexural stiffness than longitudinal flexural stiffness. Thus the f parameter would be increased and the thicker deck would exhibit better load distribution characteristics.

7. From the definition of the r parameter associated with Charts A and C, ie.,

$$r = \frac{\textit{torsional stiffness of slab beams}}{\textit{transverse flexural stiffness of slab}}$$

and from (5) above, increasing deck thickness (other parameters held constant) would result in identical increases in torsional and flexural stiffnesses of the slab. However, since torsional stiffness of the slab is only a part of the numerator of r, the r ratio would decrease and yield a larger J value and poorer load distribution characteristics (this is assuming that f remains constant).

8. Since increasing the deck thickness (while holding other parameters constant) would result in an increase in the f ratio and a decrease in the r ratio, it is not clear if the thicker deck would result in a better or poorer load distribution. This needs to be explored in more detail in a quantitative manner.

9. For all parameters other than the girder spacing, ℓ, held constant, from Equation (4.5) for f, ie.,

$$f = 0.01 \, \frac{d^3}{\ell^3} \, x \, \frac{L^4}{I}$$

we can see that f decreases inversely with the cube of ℓ. Also, from Equation (4.6) for r, ie.,

$$r = 25 \, \frac{\ell}{d^3} \, x \, \frac{C}{L^2}$$

we can see that r increases directly with ℓ. Since the rate of decrease of f w.r.t. ℓ is far greater than the rate of increase of r w.r.t ℓ, better load distribution should be attained with smaller girder spacings. Similarly, from inspection of the two equations above for f and r, b etter load distributions should be attained with longer span (L) bridges. Bridge span lengths are dictated by more important parameters than girder load distributions; however, such distributions are valid considerations in finalizing the girder spacing.

10. Comparing W steel girders and prestressed AAŞHTO girders as the supporting girders (with other parameters held constant) would yield a smaller value of the f ratio for the stiffer concrete girders. This would imply a poorer load distribution for the concrete girders. However, the concrete girders, with their greater torsional stiffness, would yield a larger value of the r ratio. This would imply a better load distribution for the concrete girders. Thus, which parameter, f or r, would dominate is not clear.

4.4 Stresses in Bridge Decks Due to Wheel Loads

In 1930 Westergaard (31) published the results of his analytical analyses of bridge deck stresses under concentrated wheel loads. He assumed the deck behaved as a homogeneous elastic slab supported on nondeflecting girders along the center lines of the girders as shown in Figures 4.17 and 4.18. Westergaard assumed the deck was either simply supported at its longitudinal edges by the girders, or was fixed at these edges. Westergaard looked at the following point load combinations.

(1) The effect of the load P_1 alone when placed at the center ($v = 0$).

(2) The combined effect at the point of application of P_1 produced by the two loads P_1 and P_2 which are separated by the definite distance a, the distance v being chosen so as to produce the greatest possible effect.

(3) The combined effect at the point of application of P_1 produced by the two loads P_1 and P_3, the definite distance b apart, when $v = 0$.

(4) The combined effect at the point of application of P_1 produced by the four loads P_1, P_2, P_3, and P_4, which are at the corners of a rectangle with dimensions a and b in the directions of x and y, the distance v being chosen so as to produce the greatest possible effect.

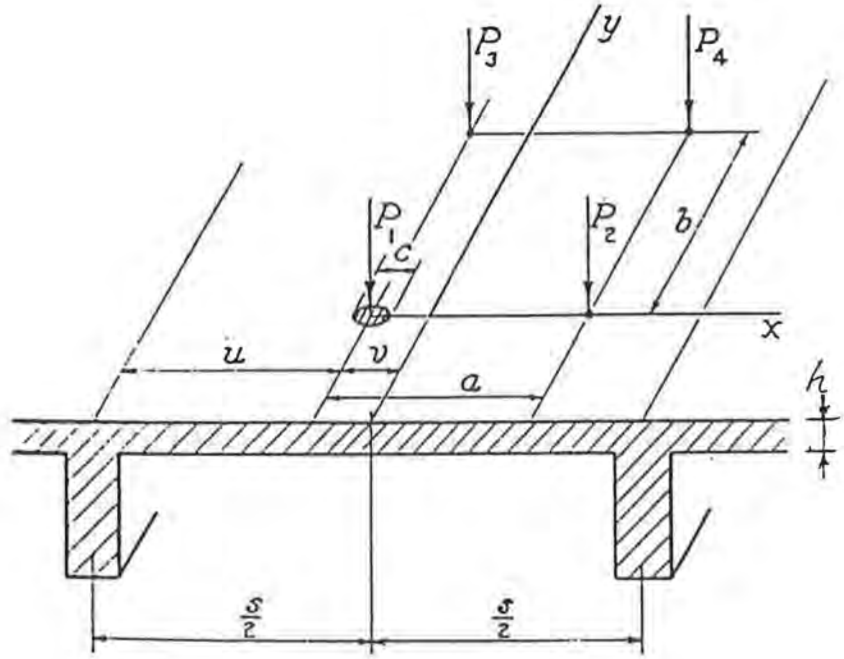


Figure 4.17 Slab Supporting Wheel Loads (31)

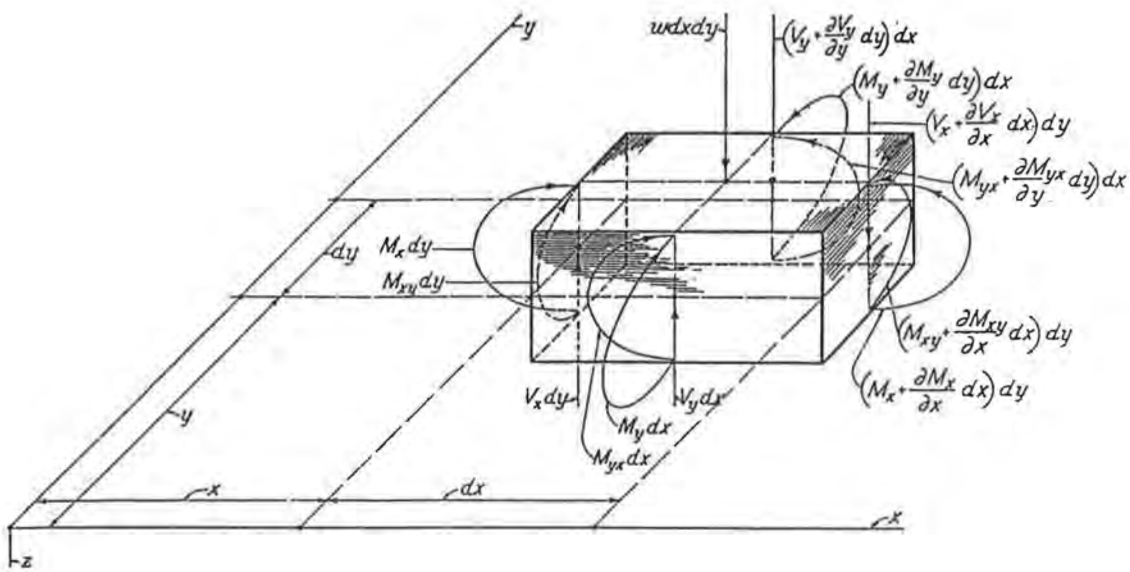


Figure 4.18 Forces and Couples Acting on Element of Slab (31)

We will only look at Westergaard's first load case, i.e., with P_1 acting alone and centered ($v = 0$) between the girders. In determining the stresses directly under the P_1 load, it will be necessary to take into consideration the fact that this load is distributed over a small area. Westergaard assumed the load to be uniformly distributed over a small circle of diameter C as indicated in Figure 4.17.

Westergaard's thin plate analysis provides bending moments M_x and M_y and twisting moments M_{xy} and M_{yx} (see Figure 4.18) per unit of width of the plate. To determine stresses at the locations of the moments, he simply divides the moment per unit width by the plate section modulus per unit width, i.e., by $\frac{h^2}{6}$, where h is the deck thickness. Thus,

$$\sigma_x = \frac{6M_x}{h^2}$$

$$\sigma_y = \frac{6M_y}{h^2} \tag{4.7}$$

$$\tau_{xy} = \frac{6M_{xy}}{h^2}$$

For the case of one wheel load at the center, Westergaard's plate theory M_x and M_y equations reduce to (the o subscript indicates values are at $v = 0$),

$$M_{0x} = \frac{P}{4\pi}\left((1 + \mu)\log_e\frac{4S}{\pi C_1} + 1\right) \tag{4.8a}$$

$$M_{0y} = M_{0x} - \frac{(1 - \mu)P}{4\pi} \tag{4.8b}$$

where, μ = Poisson's ratio

C_1 = An equivalent diameter of the circle over the area of which the load P is considered to be uniformly distributed.

h = slab thickness

When C < 3.45 h, C_1 is taken as,

$$C_1 = 2(\sqrt{0.4C^2 + h^2} - 0.675h) \tag{4.9}$$

Using this expression for C_1 and taking Poisson's ratio, $\mu = 0.15$, Equations (4.7) and (4.8) reduce to

$$M_{0x} = 0.21072\ P\left[\log_{10}\frac{S}{h} - \log_{10}\left(\sqrt{0.4\frac{C^2}{h^2} + 1} - 0.675\right) + 0.1815\right] \tag{4.10a}$$

$$M_{0y} = M_{0x} - 0.0676\ P \tag{4.10b}$$

Or,

$$\frac{M_{0x}}{P} = 0.21072\left[\log_{10}\frac{S}{h} - \log_{10}\left(\sqrt{0.4\frac{C^2}{h^2} + 1} - 0.675\right) + 0.1815\right] \tag{4.11a}$$

$$\frac{M_{oy}}{P} = \frac{M_{0x}}{P} - 0.0676 \tag{4.11b}$$

Table 4.2 and Figure 4.19 show values of the coefficient $\frac{M_{0x}}{P}$ computed from Equation (4.11a). The coefficients stated are pure numbers. If, for example, one reads in Figure 4.19, $\frac{M_{0x}}{P} = 0.3$, the significance is: $M_{0x} = 0.3P$, or, with P = 10,00 pounds, M_{0x} = 0.3 x 10,000 pounds = 3,000 pounds = $3{,}000\ \frac{in.\ lbs.}{in.} = 3{,}000\ \frac{ft.\ lbs.}{ft.}$ (the unit of bending moment per unit of width being inch-pounds per inch or foot-pounds per foot or simply pounds). If units of the metric system were used, the coefficients in Figure 4.19 would remain unchanged.

Table 4.2 Values of the coefficient M_{0x}/P of the maximum bending moment per unit of width, produced at the center of the slab in the direction of the span by a central load P distributed uniformly over the area of a small circle with diameter C. The edges are assumed to be simply supported. The values were computed from Equation 5a for different relative values of the span, S, the thickness, h, and the diameter C. Poisson's ratio, $\mu = 0.15$ (31).

S	$c=0$	$c=0.05s$	$c=0.10s$	$c=0.15s$	$c=0.20s$	$c=0.25s$
$s=6h$	0.3051	0.3003	0.2874	0.2701	0.2520	0.2345
$s=8h$	.3315	.3230	.3026	.2784	.2552	.2345
$s=10h$	.3519	.3390	.3110	.2811	.2550	.2326
$s=12h$	.3685	.3508	.3154	.2815	.2535	.2303
$s=14h$	.3827	.3595	.3178	.2809	.2516	.2284
$s=16h$	.3949	.3660	.3186	.2798	.2499	----------
$s=18h$	.4056	.3709	.3186	.2786	----------	----------
$s=20h$	.4153	.3744	.3184	.2771	----------	----------

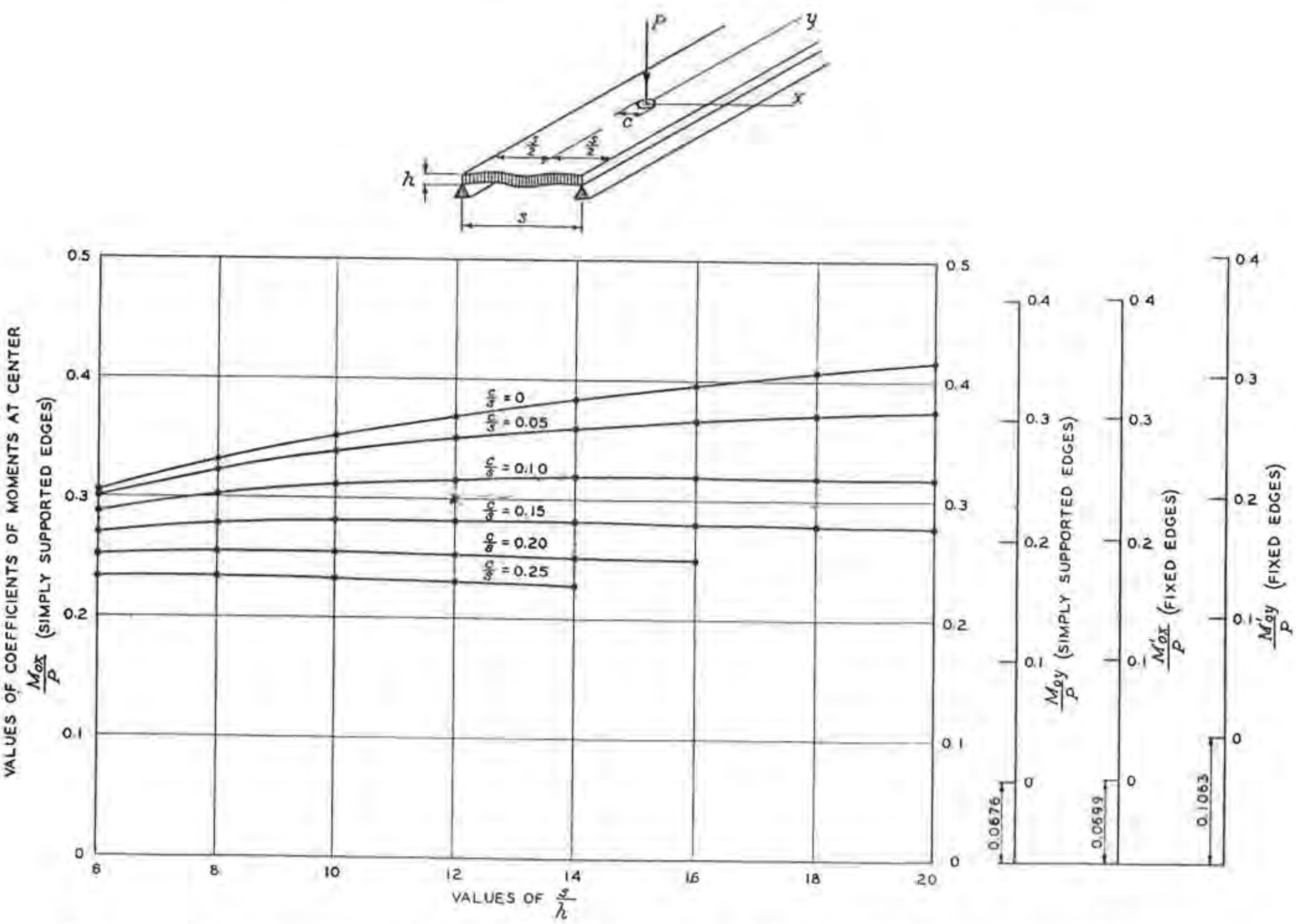
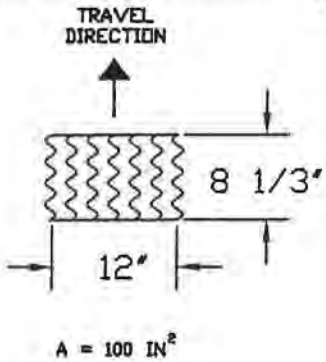


Figure 4.19 Coefficients of Bending Moments, M_{0x} and M_{0y}, in Directions of x and y, Respectively Produced at Center of Slab by a Central Load P Distributed Uniformly Over the Area of a Small Circle with Diameter C. Poisson's ratio, $\mu = 0.15$ (31).

Since the difference between $\frac{M_{0x}}{P}$ and $\frac{M_{0y}}{P}$ is constant, the curves in Figure 4.19 also represent values of $\frac{M_{0y}}{P}$ as indicated on the third scale from the right.

Example Case 1.



Assume:

a) A tire print of →
b) $S = 96"$
c) $h = 8"$
d) $P = 16{,}000$ lb
e) $f'_c = 4000$ psi

Therefore,

$$\frac{\pi C_1^2}{4} = 100\ in^2$$

$C_1 = 11.28"$

Using Equation (4.9),

$$C_1 = 2\left(\sqrt{0.4C^2 + h^2} - 0.675h\right)$$

$$11.28" = 2\left(\sqrt{0.4C^2 + 8^2} - 0.675(8)\right)$$

$C = 12"$

$$\therefore \frac{C}{S} = \frac{12}{96} = 0.125 \rightarrow C = 0.125S$$

$$\frac{s}{h} = \frac{96}{8} = 12 \rightarrow S = 12h$$

Using linear interpolation, from Table 4.2 (or from Fig.4.19),

$$\frac{M_{0x}}{P} = 0.2985$$

$M_{0x} = 0.2985P$

From Equation (4.11b),

$$\frac{M_{0y}}{P} = 0.2985 - 0.0676 = 0.2309$$

$$M_{0y} = 0.2309\ P$$

$$\therefore M_{0x} = 0.2985\ (16{,}000\ lbs) = 4{,}776\ in.\ lbs./in.$$

$$\sigma_{0x} = \frac{6M_{0x}}{h^2} = \frac{6 \times 4{,}776}{(8)^2 in^2} = 448\ psi$$

$$M_{0y} = 0.2309\ (16{,}000\ lbs.) = 3{,}694\ in.\ lbs./in.$$

$$\sigma_{0y} = \frac{6M_{0y}}{h^2} = \frac{6 \times 3{,}694}{(8)^2 in^2} = 346\ psi$$

Using the ACI recommended value of tensile strength of concrete as

$$f_t' = f_r = 7.5\sqrt{f_c'}$$

$$= 7.5\sqrt{4000} = 474\ psi$$

Therefore, the deck will not crack (on the underside) under the wheel load.

Example Case 2.

Assume:

a) Tire print is same as Case 1

b) S = 96"

c) h = 5.61" (average deck thickness in UAB Study)

d) P = 16,000 lb

e) f'_c = 3,000 psi

Again C_1 = 11.28"

Using Equation (4.9),

$$11.28" = 2\ (\sqrt{0.4C^2 + 5.61^2} - 0.675(5.61))$$

C = 12"

$$\therefore \frac{C}{S} = \frac{12}{96} = 0.125 \rightarrow C = 0.125S$$

$$\frac{S}{h} = \frac{96}{5.61} = 17.11 \rightarrow S = 17.11h$$

From Figure 4.19,

$$\frac{M_{0x}}{P} = 0.298$$

$$M_{0x} = 0.298P = 0.298\ (16{,}000\ lbs) = 4{,}768\ in.\ lbs./in.$$

$$\sigma_{0x} = \frac{6M_{0x}}{h^2} = \frac{6 \times 4{,}768}{(5.61)^2\ in^2} = 909\ psi$$

$$\frac{M_{0y}}{P} = 0.298 - 0.0676 = 0.2304$$

$$M_{0y} = 0.2304\ P = 0.2304\ (16{,}000) = 3{,}686\ in.\ lbs./in.$$

$$\sigma_{0y} = \frac{6 \times 3{,}686}{(5.61)^2\ in^2} = 703\ psi$$

$$f_t' = f_r = 7.5\sqrt{3{,}000} = 411 psi$$

Therefore, the deck will crack on the underside under the wheel load in both the x and y directions. This is consistent with what we are observing on the underside of the Birmingham interstate bridge decks.

Note, in comparing Example Cases 1 and 2 results for C, we see that Equation (4.9) is not very sensitive to the deck thickness, h. Example Case 1 used an h = 8" and Example Case 2 used an h = 5.61", and they both produced a C value of 12" or the same $\frac{C}{S}$ value. Thus, this coupled with an inspection of Figure 4.19 which shows very small changes in slab moment ($\frac{M_{0x}}{P}$) over a large range of $\frac{S}{h}$ values ($10 < \frac{S}{h} < 20$) for a given value of $\frac{C}{S}$ (except for $\frac{C}{S} = 0$ which is not a realistic value), indicates that the deck bending moment due to wheel loadings would be almost independent of the deck thickness. Then from Equation (4.7), we see that the bending stresses vary inversely with the square of the deck thickness just as with simple 1-way slab action theory.

Thus the propensity for deck cracking on the underside from wheel loads are inversely proportional to the square of the deck thickness. The propensity for cracking relative to the often used (in the past) deck thickness of 6¼" are indicated in Table 4.3. It can be noted in Table 4.3 that increasing the deck thickness to 7" significantly decreased the propensity for cracking under wheel loads; however, if the thickness were increased to 8", it would result in a dramatic decrease in propensity for cracking under wheel loads.

It can be noted that in addition to causing structural deck cracking, truck wheel loads are also the source of deck fatigue damage and deterioration. Since concrete does not have an endurance limit like steel, each truck wheel load application does some fatigue damage which is proportional to the stress level. Since deck stresses due to wheel loads vary inversely as the square of the deck thickness, then the relative propensities for cracking shown in Table 4.3 are probably, to some scale, the relative propensities for fatigue damage from truck wheel loadings.

Thus, thicker decks greatly reduce tensile cracking under truck wheel loadings and also greatly reduce deck fatigue damage under these same loadings. Both of these reductions should translate into improved durability and service life for the deck.

Table 4.3. Propensity for Deck Cracking from Wheel Loads Relative to Deck Thicknesses of 6¼" and 6".

Deck Thickness (h) (in)	$1/h^2$ (in^{-2})	Relative Propensity for Cracking $(1/h^2)/(1/h^2_{6.25})$	$(1/h^2)/(1/h^2_{6"})$
6	0.02778	1.085	1.00
6¼	0.02560	1.000	-
7	0.02041	0.797	0.73
8	0.01563	0.610	0.56
9	0.01235	0.482	0.44

4.5 Deck Punching Shear

An uncracked bridge deck resists traffic load primarily by one-way (transverse) flexure, with in-plane forces being fairly insignificant. However, after the deck is significantly cracked, it resists traffic loads primarily through arching action, similar to a flat dome. Arching action is defined by a zone of compression radiating out from the point of load and a surrounding zone of circumferential tension (hoop stresses) in equilibrium with the radiating compressive forces as illustrated in Figure 4.20. These compressive membrane forces increase the flexural capacity of a bridge deck (15).

The AASHTO and ACI punching shear capacity equations for the punching shear capacity associated with a rectangular footprint in nonprestressed slabs and for slabs prestressed in one direction only are identical, and can be expressed as (15):

$$V_c = 2\,(b_1 + b_2 + 2\bar{d})\,\bar{d}\,f_t \tag{4.12}$$

$$f_t = \left(2 + \frac{4}{\beta_c}\right)\sqrt{f'c} \le 4\sqrt{f'c} \tag{4.13}$$

where:

V_c = punching shear capacity, lbs.

b_1 = short side of concentrated load or reaction area, in.

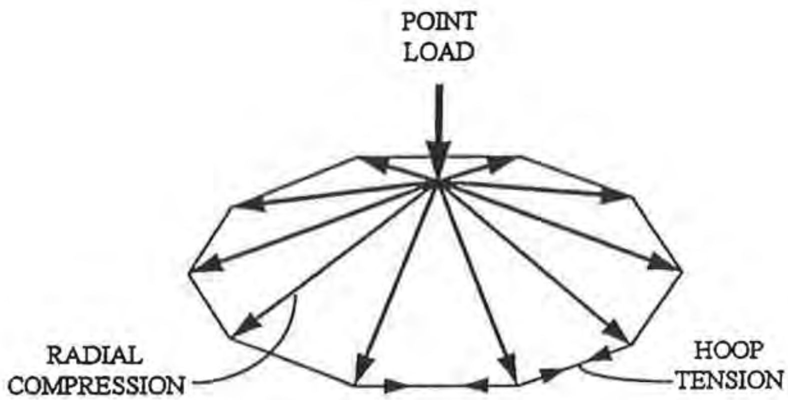
b_2 = long sides of concentrated load or reaction area, in.

$\bar{d}$ = average effective depth of section, in.

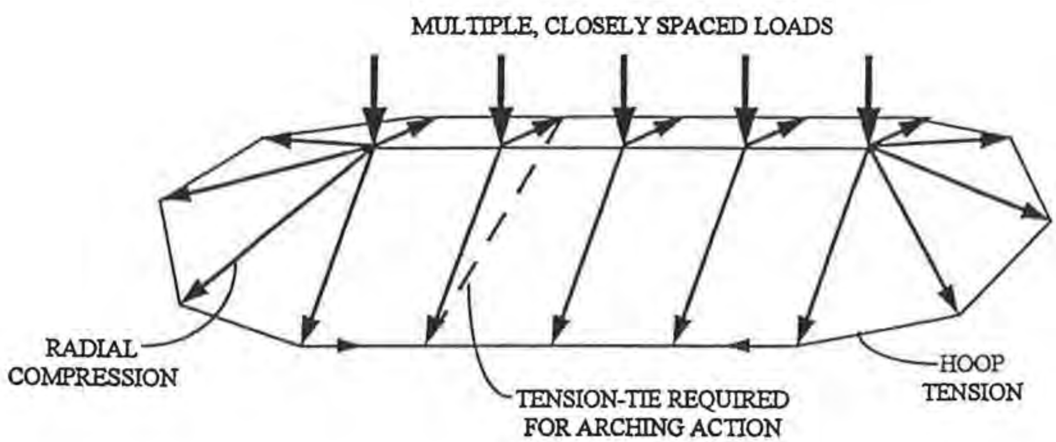
f_t = ultimate tensile capacity of concrete, psi

$\beta_c = \dfrac{b_2}{b_1}$

$f'c$ = specified compressive strength of concrete, psi



a.) Single Point Load



b.) Multiple Closely Spaced Loads

Figure 4.20 Arching Action in Concrete Slab Due to Point and Multiple Loads (15)

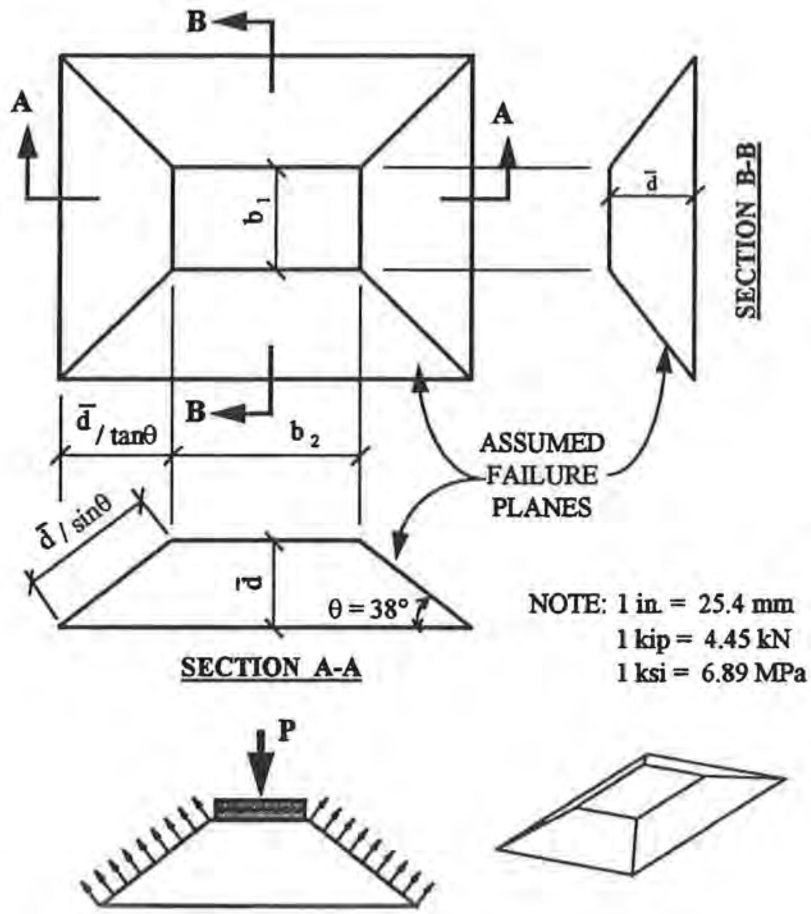


Figure 4.21 General Punching Shear Model Showing Assumed Failure Planes and Forces Acting in Equilibrium (15).

In their work (15), Graddy, et al. used the general punching shear model shown in Fig. 4.21. Working with this model and equilibrium of force requirements yields the following punching shear capacity equation

$$V_c = 2\left(b_1 + b_2 + \frac{2\bar{d}}{\tan\theta}\right)\frac{\bar{d}}{\tan\theta} f_t \tag{4.14}$$

where:

$\theta = 38°$ as shown in Fig. 4.21

f_t = that given by Eqn. 4.13

It should be noted that Eqn. 4.14 reduces to the AASHTO and ACI equation when θ is set to 45°.

4.6 Approximate LL Deflections,Girder Spacings and L/D Ratios

Maximum bridge deck LL deflections can be modeled and assessed using one of the three modeling/formulations shown in Table 4.4. For a bridge deck on multigirders (3 or more girders), the edge deck span is the most flexible and can be conservatively modeled for LL deflection evaluation as shown in Case 2 in Table 4.4. Rounding off we will assume,

$$\Delta_{MAX}^{LL} = 0.0100 \frac{PL^3}{EI} \tag{4.15a}$$

Also, we will assume

$$\Delta_{MAX\ ALLOWED}^{LL} = \frac{L}{1000} \tag{4.15b}$$

Therefore,

$$0.0100 \frac{PL^3}{EI} \leq \frac{L}{1000} \tag{4.15c}$$

Table 4.4 Bridge Deck Modelling and Formulas for LL Deflection Assessment

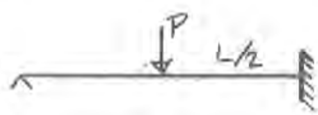
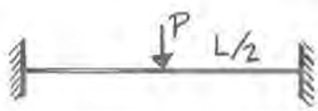
Case	BC and Loading Modelling	Maximum Deflection
1		$\Delta_{MAX} = \dfrac{PL^3}{48\ EI} = 0.02083\ \dfrac{PL^3}{EI}$
2		$\Delta_{MAX} = 0.009317\ \dfrac{PL^3}{EI}$
3		$\Delta_{MAX} = \dfrac{PL^3}{192\ EI} = 0.00521\ \dfrac{PL^3}{EI}$

Table 4.5. Maximum L and L/D for Various Bridge Deck Thickness Based on LL Deflection Limitations

Deck Thickness (D) (in)	Maximum Girder Spacing (L) (in)	Maximum (L/D)
6.5	78.7	12.10
7.0	87.9	12.56
7.5	97.5	13.00
8.0	107.4	13.43
8.5	117.6	13.84
9.0	128.2	14.24

Working with a 1-foot width of deck yields,

$$0.0100 \frac{PL^2 \cdot L}{E \dfrac{12'' D^3}{12}} \leq \frac{L}{1000}$$

$$0.0100 \frac{DL^2}{E\ D^3} \leq \frac{1}{1000}$$

$$0.0100 \frac{PL}{E\ D^2} \frac{L}{D} \leq \frac{1}{1000}$$

Or,

$$\frac{L}{D} \leq \frac{ED^2}{10PL} \qquad (4.15d)$$

Let us now examine L_{MAX} and $(L/D)_{MAX}$ values for various deck thicknesses (D) and for

$P = 16^k$ (truck design wheel load)

$E = 57{,}000\sqrt{4000} = 3{,}605$ k/in^2

For D = 8"

$$\frac{0.0100 \ x\ 16\ x\ L^2}{3{,}605\ x\ 512} \leq \frac{1}{1000}$$

$$L^2 \leq \frac{.001\ x\ 3605\ x\ 512}{.010\ x\ 16}$$

$L^2 \leq 11{,}536$

$L \leq 107.4"$ (8.95')

$$(\frac{L}{D})_{MAX} = \frac{107.4}{8} = 13.43$$

Repeating these computations for D = 6.5 to 9.0 inches, yields the L_{MAX} and $(\frac{L}{D})_{MAX}$

values shown in Table 4.5 and plotted in Figure 4.22. Note that the AASHTO requirement regarding L_{max} given in Chapter 2 is shown superimposed on Fig. 4.22.

4.7 Transverse Vibrations of Plates/Decks

Figure 4.23 shows a plate with a uniform thickness h that is assumed to be small in comparison with its other dimensions. Take the x-y plane as the middle plane of the plate and assume that deflections in the z direction are small in comparison with the thickness h.

In the case of a rectangular plate (see Figure 4.23a) with simply supported edges, Timoshenko (29) proceeds to take the deflection of the plate during vibration as the double series

$$v = \sum_{m=1}^{\infty} \sum_{n=1}^{\infty} \phi_{mn} \sin \frac{m\pi x}{a} \sin \frac{n\pi y}{b} \tag{4.16}$$

which are the normal functions for the case under consideration. It is easy to see that each term of this series satisfies the conditions at the edges, which require that

$v = \partial^2 v/\partial x^2 = 0$ at $x = 0$ and $x = \alpha$; and $v = \partial^2 v/\partial y^2 = 0$ at $y = 0$ and $y = b$.

The inertial force for a typical element of the plate is $-\rho h\ddot{v}\, dx\, dy$, where ρh is the mass per unit area. The differential equation of motion for free vibrations in principal coordinates is

$$\rho h \ddot{\varphi}_{mn} + \pi^4 D\varphi_{mn} \left(\frac{m^2}{a^2} + \frac{n^2}{b^2} \right)^2 = 0 \tag{4.17}$$

The solution of this equation is

$$\varphi_{mn} = C_1 \cos pt + C_2 \sin pt \tag{4.18}$$

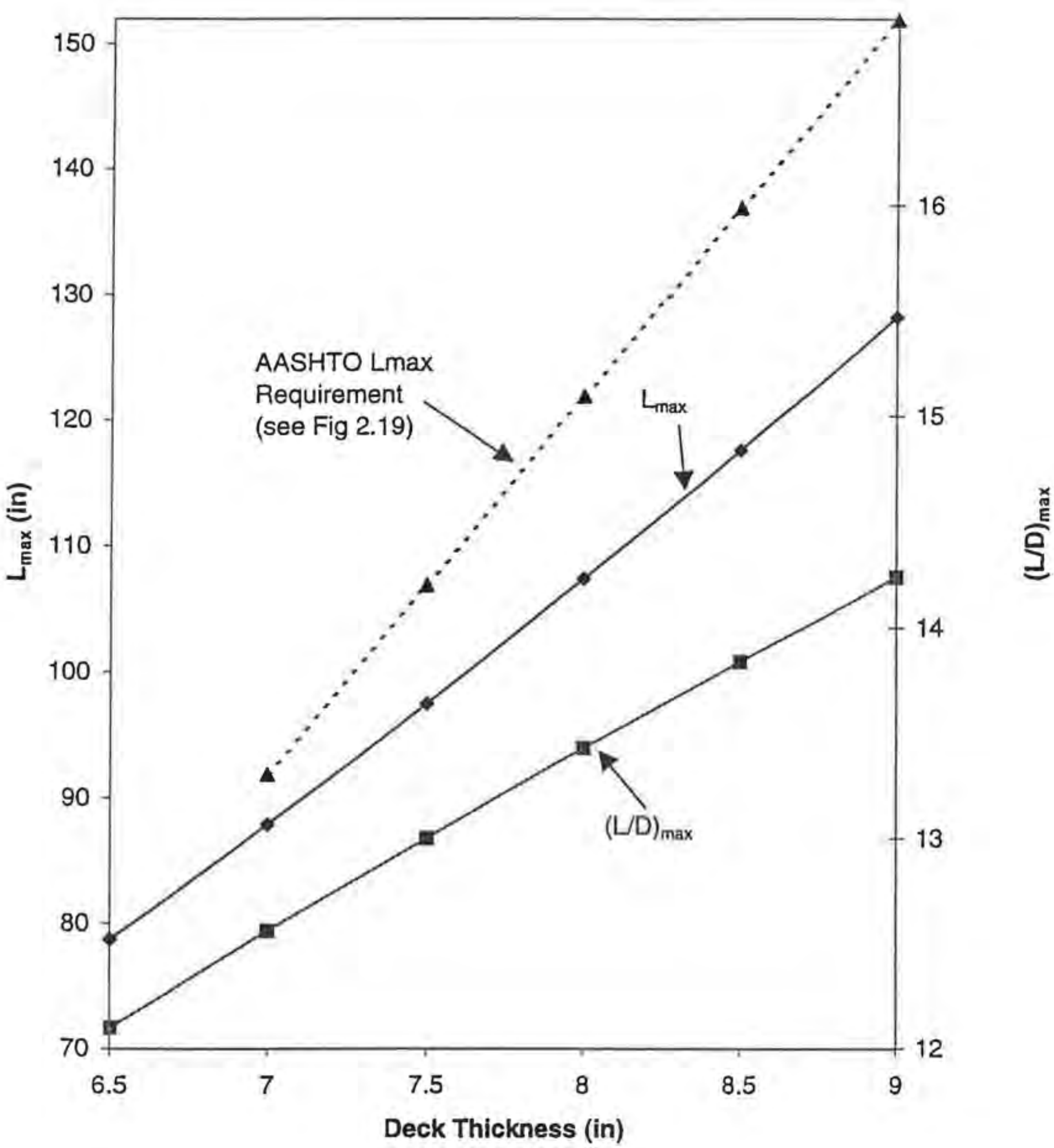


Fig. 4.22 L_{MAX} and $(\frac{L}{D})_{MAX}$ v.s. Deck Thickness (D) - Based on LL Deflection Limitations

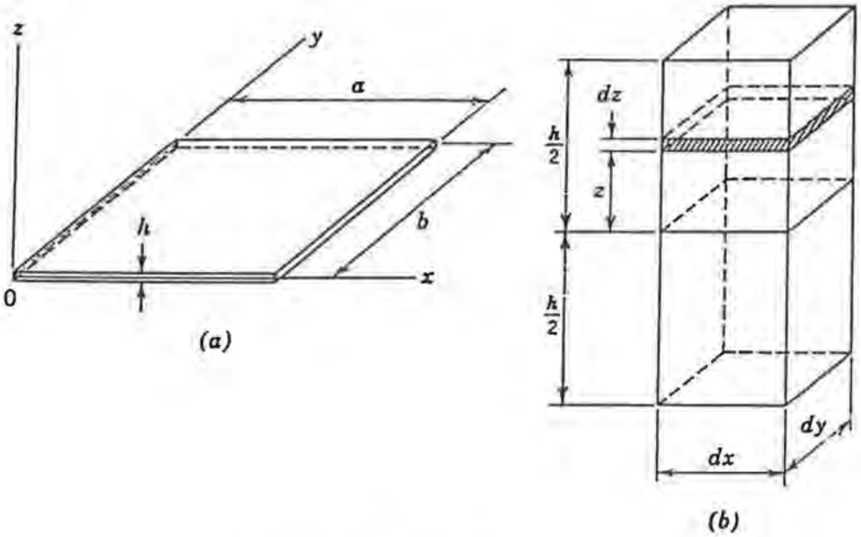


Fig. 4.23. Thin Plate (29)

where p is the circular frequency of vibration (in radians per second), and is given by

$$p = \pi^2 \sqrt{\frac{D}{\rho h}} \left(\frac{m^2}{a^2} + \frac{n^2}{b^2} \right) \tag{4.19}$$

where $D = Eh^3/[12(1 - \nu^2)]$ is the flexural rigidity of the plate.

In the case of a square plate, we obtain for the lowest mode of vibration,

$$p_1 = \pi^2 \sqrt{\frac{D}{\rho h}} \left(\frac{1}{a^2} + \frac{1}{a^2} \right) = \frac{2\pi^2}{a^2} \sqrt{\frac{D}{\rho h}} \tag{4.20}$$

Or, in terms of cyclic frequency,

$$f_1 = \frac{p_1}{2\pi} = \frac{\pi}{a^2} \sqrt{\frac{D}{\rho h}} \quad \text{(cps or hertz)} \tag{4.21}$$

For other boundary conditions, for square plates, the frequencies of the various modes of vibration can be determined by

$$p = \frac{\alpha}{a^2} \sqrt{\frac{D}{\rho h}} \tag{4.22}$$

in which α is a constant depending on the mode.

From Equations (4.20-4.22) we see that the natural frequencies of vibration vary inversely with the square of the plate span, a. Thus, as girder spacings increase, say from 6 feet to 9 feet, the normalized deck fundamental natural frequency and period of vibration would decrease and increase respectively as shown in Figure 4.24. In general, the lower deck natural frequencies for the larger girder spacings are not good as they move the deck fundamental frequency closer to those of the suspension systems of large tractor trucks using the bridge.

Substituting the expression for D into Equation (4.21), i.e.,

$$f_1 = \frac{\pi}{a^2}\sqrt{\frac{Eh^3}{12(1-\nu^2)\rho h}} = \frac{\pi h}{a^2}\sqrt{\frac{E}{12(1-\nu^2)\rho}} \tag{4.23}$$

we see that the natural frequencies of vibration of a plate or deck vary directly with the deck thickness, h. This is shown graphically in Figure 4.25. The increase in deck fundamental frequency with thickness in general is good if one is looking to increase deck thickness as this will move the deck frequency further away from those of the suspension systems of large trucks.

Rather than behaving as a flat plate with idealized support conditions as indicated above, a deck-girder bridge superstructure probably behaves more like a deck-girder floor system. Rogers (27) considered the vibration characteristics of a simple floor system, and his modelling and results are presented below.

Consider a floor system consisting of a single girder simply supported at its ends by a rigid wall and supporting along its length a series of parallel transverse floor beams. For convenience all members are assumed to be of uniform properties throughout their lengths. Figure 4.26 illustrates the pertinent dimensions of the system.

The properties of the girder are characterized by the subscript 0, whereas the properties of the floor beams are denoted without subscripts except to indicate which of a group is under consideration.

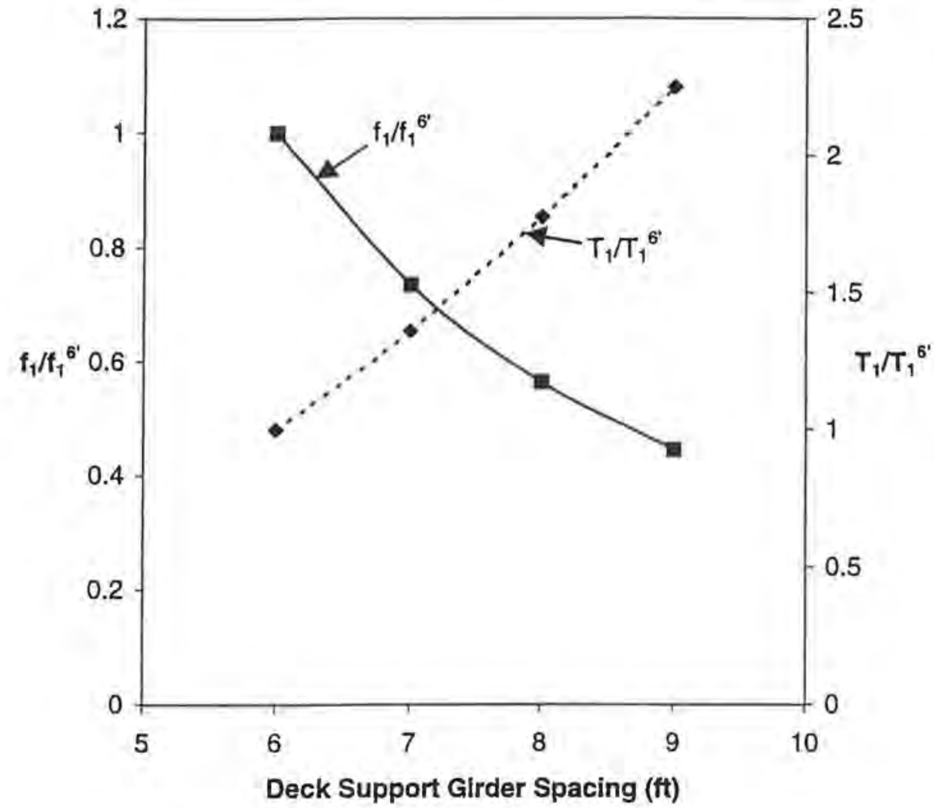


Fig. 4.24 Normalized Frequency and Period of Vibration vs. Girder Spacing for Thin Plate Modelling of Deck

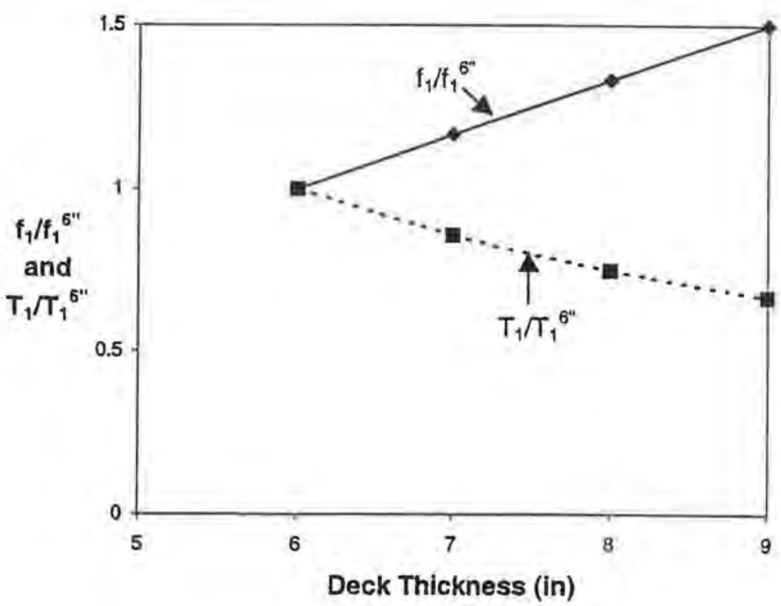


Fig. 4.25 Normalized Frequency and Period of Vibration vs. Deck Thickness for Thin Plate Modelling of Deck

When the structure is subjected to a free vibration all points move with the same frequency of oscillation. Let the frequency of the nth mode of free vibrations be p_n, and let the deflection of the girder at the point of support for the floor beam be given by

$$y_0 = Y_j \sin p_n t \tag{4.24}$$

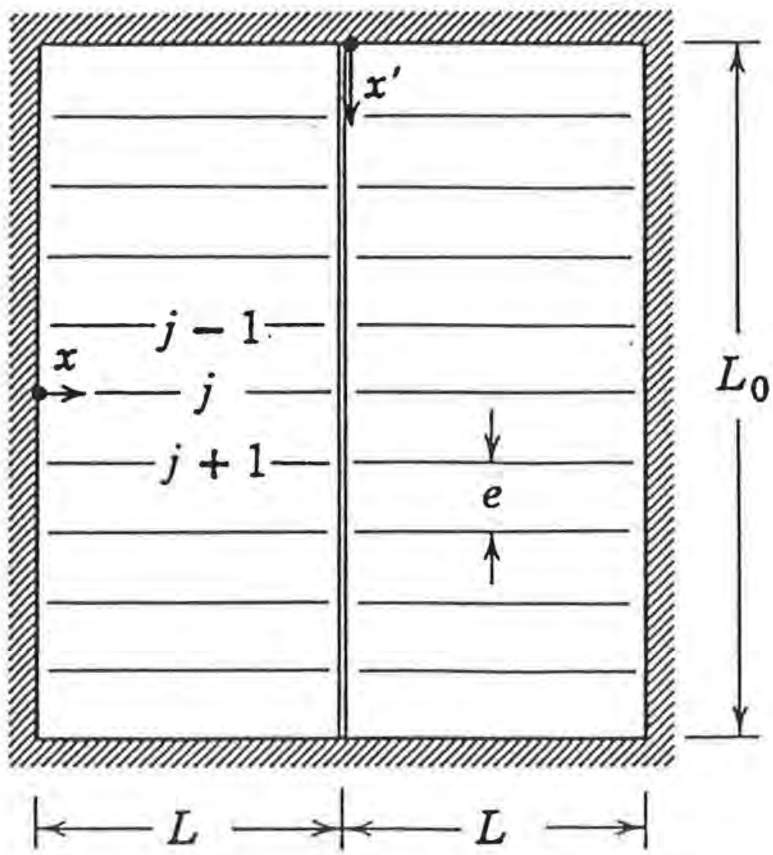


Fig. 4.26. Simple Beam-Girder Floor System (27)

The end of the floor beam which is supported by the girder undergoes the same displacement, y_0. The motion of the floor beam is governed by the basic equation

$$EI\frac{\partial^4 y}{\partial x^4} + m\frac{\partial^2 y}{\partial t^2} = 0 \tag{4.25}$$

and during the free vibration in the nth mode the structure will have the deflection at any point given by

$$y(x,\ t) = X_n \sin p_n t \tag{4.26}$$

where end conditions on X_n are given by

$$X_n(0) = X''_n(0) = X''_n(L) = 0 \tag{4.27}$$

$$X_n(L) = Y_j$$

If all of the floor beams have the same design and if the beam spacing e (see Fig.4 26) is not too large, we may wish to approximate the load brought to the girder from the floor beams by a distributed load rather than a series of concentrated loads.

After some work, Rogers determines the frequency (circular) equation for the free vibrations of the floor system to be

$$p_n^2 = n^4 p_0^2 + \frac{EI\lambda_n^3}{m_0 e}(\cot \lambda_n L - \coth \lambda_n L) \tag{4.28}$$

where n designates the particular mode, and

$$p_0^2 = \pi^4 E_0 I_0 / m_0 L_o^4 \tag{4.29}$$

$$\lambda_n^4 = \frac{m p_n^2}{EI} \tag{4.30}$$

An alternate form of Equation (4.28), which is convenient in finding the roots $\lambda_n L$, is

$$(\lambda_n L)^4 = n^4\pi^4\left(\frac{mL^4}{m_0 L_0^4}\right)\left(\frac{E_0 I_0)}{EI}\right) + \left(\frac{mL}{m_o e}\right)\left[\frac{\cot \lambda_n L - \coth \lambda_n L}{(\lambda_n L)^{-3}}\right] \tag{4.31}$$

Assume that the floor system shown in Figure 4.26 is composed of steel beams and steel girders with dimensions of

$$\frac{L}{L_0} = \frac{1}{5};\ \ \frac{I_0}{I} = 10;\ \ \frac{E_0}{E} = 1;\ \ \frac{L}{e} = 4;\ \ e = 5\ ft. \tag{4.32}$$

and that the floor beams weigh 50 ppf and the girder weighs 160 ppf. The dead weight of the concrete slab which rests on the floor beams plus the distributed live load is assumed to be 310 psf. This load is assumed to be carried directly by the floor beams.

The weight carried by the floor beams per foot of length is given by

$$310e + 50 = 1600 \text{ ppf} \tag{4.33}$$

The weight carried by the girders per foot of length is 160 ppf. The ratio of the masses is, therefore,

$$\frac{m}{m_0} = \frac{1600/g}{160/g} = 10 \tag{4.34}$$

where g is the gravitational constant.

The frequency equation (Eqn. (4.31)) for this floor system becomes

$$(\lambda_n L)^4 = 15.586n^4 + 40\left[\frac{\cot \lambda_n L - \coth \lambda_n L}{(\lambda_n L)^{-3}}\right] \tag{4.35}$$

For the fundamental mode of vibration, n = 1, and for this case the value $\lambda_1 L = 0.866$. From Equation (4.30),

$$p_1 = 0.75 \sqrt{EI/mL^4} \tag{4.36}$$

or,

$$p_1 = 1.875 \sqrt{E_0 I_0/m_0 L_0^4} \tag{4.37}$$

The period for the fundamental mode is

$$T_1 = 3.35\sqrt{m_0 L_0^4/E_0 I_0} \tag{4.38}$$

or,

$$T_1 = 8.378 \sqrt{mL^4/EI} \tag{4.39}$$

For the case of a concrete-girder bridge deck/superstructure system, the transverse floor beams of the above floor system could be 1- foot widths of the concrete deck. As can be seen in Equation (4.36) the system natural frequency of vibration varies directly with the deck thickness, h, as was the case for the thin plate modelling of the deck discussed earlier.

It can be noted that Itani (17) indicates that first mode frequencies and periods of vibration for simple span steel girder highway bridges are typically around 1 hz and 1 sec respectively, while for concrete girder bridges they are typically around 3-4 hz and 0.33 - 0.25 sec. Thus, typical steel girder superstructure bridges have fundamental natural frequencies (≈ 1 hz) which are quite close to those of the suspension systems of large tractor trucks (≈ 0.5 - 1.0 hz). This is consistent with the writer's observations from standing on the decks of different bridges as large tractor trucks pass by.

4.8 Deck Design

Slabs in slab-girder bridges can be designed by three methods, i.e.,

- the analytical strip method,
- the empirical method, and
- the yield-line method.

The three methods yield different deck slab designs which are generally similar but different. Each method is briefly discussed below.

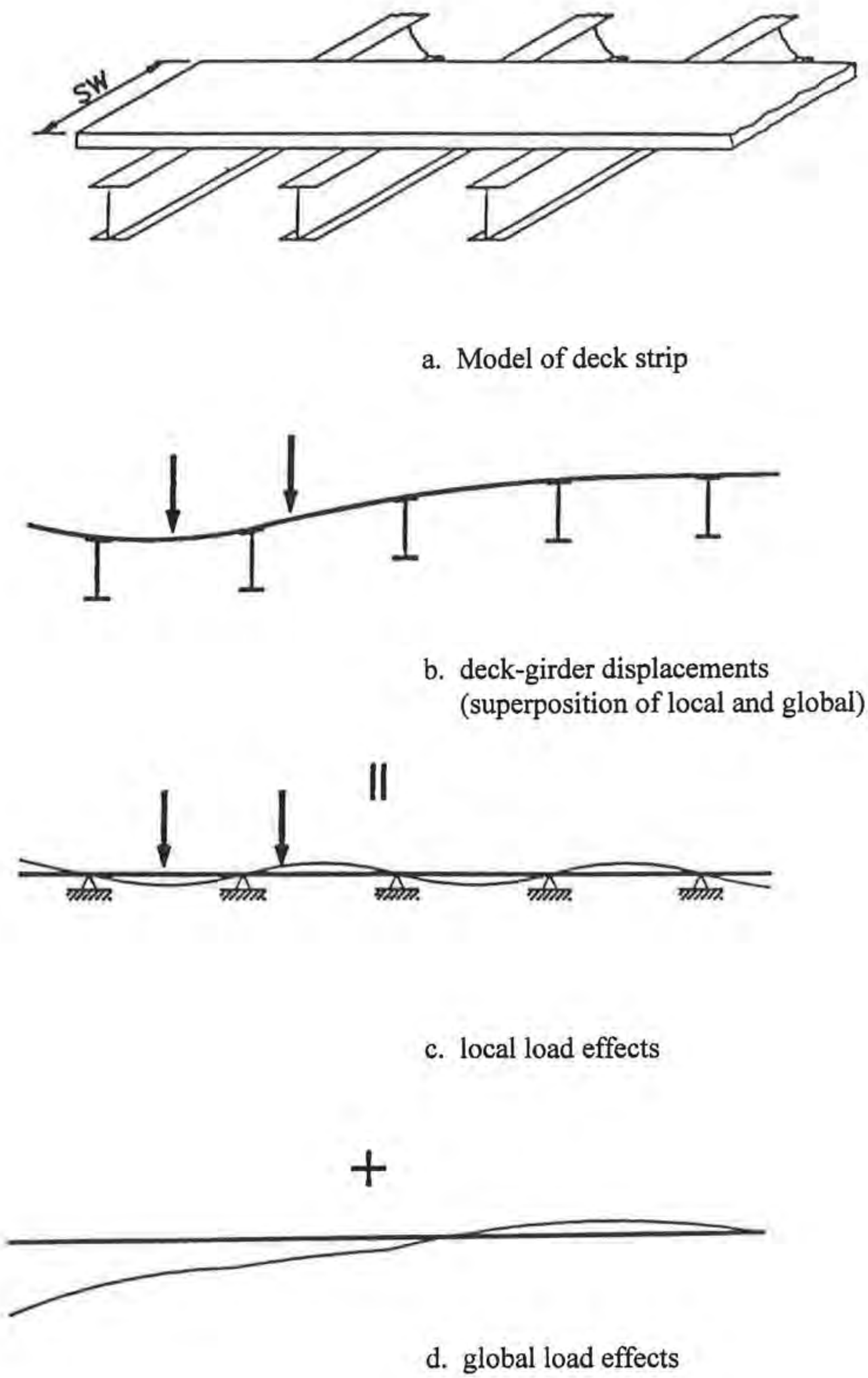
Analytical Strip Method. A deck slab may be considered as a one-way slab because its aspect ratio (panel length/panel width) is large (greater than 1.5). Therefore, assuming the deck load is carried to the support girders by one-way action, a primary question is "what is the appropriate width of the 1-way strip, i.e., strip width (SW)?" Figure 4.27 graphically illustrates the deck behavior assumed in the strip design method. It should be noted in Figure 1 that for decks, the local load effects are usually significantly greater than the global effects, and thus the deck can be analyzed and design assuming the girders provide nonsettling supports as indicated in Fig. 4.27c.

It can be noted that in deck design by the strip method, local effects dominate and truck wheel loads would be the critical loadings for shear, moment, and deflections. In this case, the 1-way deck slab has

V_{max}^{LL} which is approximately independent of girder spacing (s)

M_{max}^{LL} which varies linearly with s.

Δ_{max}^{LL} which varies as the cube of s and inversely as the cube of the deck thickness t, i.e., $\Delta_{max} = f(\frac{s^3}{t^3})$.



a. Model of deck strip

b. deck-girder displacements (superposition of local and global)

c. local load effects

d. global load effects

Fig. 4.27. Strip Method Assumed Behavior (6)

Therefore, at service load levels,

$$\sigma_{max}^{LL} = \frac{M_{max}^{LL}\, y}{I} = f(\frac{s}{t^2})$$

and at ultimate load,

$$M_{ult} = A_s\, f_y\, (d - \frac{a}{2}) = A_s\, f_y\, (d - \frac{A_s\, f_y}{0.85 f_c' b \cdot 2})$$

For a safe design,

$$M_{ult} \geq factored\ M_{max\ applied} \qquad (or\ f(t) \geq f(s))$$

or,

$$\frac{M_{ult}}{factored\ M_{max\ applied}} \geq 1$$

or,

$$f(\frac{t}{s}) \geq 1$$

Thus, for a bridge deck, for a given deck thickness, t, making the girder spacing, s, smaller is better, or for a given girder spacing, s, making the deck thickness, t, larger is better.

Yield-Line Method. In the yield-line method, the slab is assumed to behave inelastically and exhibit adequate ductility to sustain the applied load until the slab reaches a collapse mechanism. Because reinforcement requirements of the AASHTO provide underreinforced or ductile deck systems, the yield-line approach is realistic. However, this method only provides information on ultimate load capacity, and the behavior of the deck under service loads must be assessed in some other manner.

The fundamentals and the primary assumptions of the yield-line theory are as follows (9):

- In the mechanism, the bending moment per unit length along all yield lines is constant and equal to the moment capacity of the section.
- The slab parts (area between yield lines) rotate as rigid bodies along the supported edges.

- The elastic deformations are considered small relative to the deformation occurring in the yield lines.
- The yield lines on the sides of two adjacent slab parts pass through the point of intersection of their axes of rotation.

Consider now a bridge deck which is subjected to concentrate truck wheel loads, P, and a uniformly distributed load, q. Assumed yield line patterns for a portion of the deck are as shown in Figure 4.28. In this figure, the girder spacing is S, the cantilever overhang is H, and G is the wheel spacing (gage), usually 6 ft , or the spacing between the wheels of adjacent trucks, usually 4 ft.

Using an energy approach where external work done by the loads is equal internal energy stored in the system yields for the load position A (see Fig. 4.28),

$$m' + m = \frac{P}{2\pi} + \frac{qr^2}{6} \tag{4.40}$$

If we assume the positive and negative moment capacities (m and m′ respectively) are equal, and neglect the uniform load, Equation (4.40) reduces to

$$m = m' = \frac{P}{4\pi} \tag{4.41}$$

For the case of two wheel loads positioned at points C in Figure 4.28, conservation of energy yields,

$$P + \frac{qr}{2}\left(\frac{\pi r}{3} + G\right) = \pi(m' + m) + \frac{G}{r}\left(m_\ell^{'} + m_\ell\right) \tag{4.42}$$

For the case of loads at point D in Fig. 4.28, the only difference between the analysis of

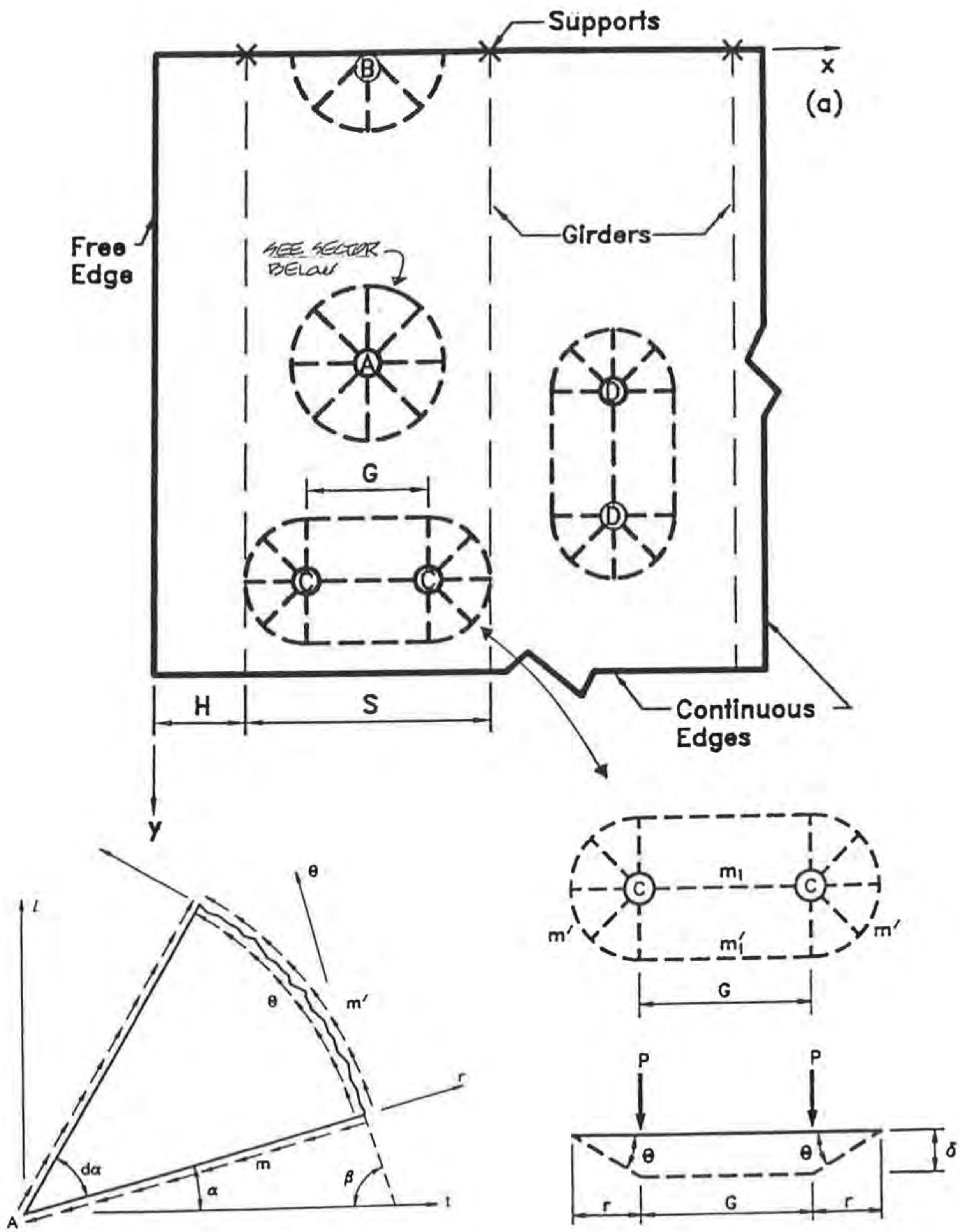


Fig. 4.28 Portion of Deck Slab Showing Load Positions (6)

this position and that of point C is that the moment capacity associated with the straight lines is now the capacity of the transverse reinforcement. This Eqn. (4.42) becomes

$$P + \frac{qr}{2}\left(\frac{\pi r}{3} + G\right) = \pi(m' + m) + \frac{G}{r}\,(m_t^{'} + m_t) \tag{4.43}$$

Note that as G goes to zero, Eqs. 4.42 and 4.43 reduce to Eq. (4.40) as expected.

Empirical Method. Recent research has shown that the primary structural action of concrete decks is not flexure, but internal arching. The arching creates an internal compressive dome, and only a minimum amount of isotropic reinforcement is required for local flexural resistance and global arching effects. With the empirical design approach there is no need for any analysis. When the design conditions listed in (1) below are met, and the dome, and the predetermined minimum reinforcement in all four layers listed in (2) below is specified, that completes the design. The empirical method is applied to the interior deck spans of the traditionally designed (strip method) deck shown in Fig. 4.29. The design steps and process are shown below and the resulting design is shown in Fig. 4.30 (6).

1. *Design Conditions*. Design depth excludes the loss due to wear, h = 190 mm. The following conditions must be satisfied:

    - Supporting component are made of steel and/or concrete, YES
    - The deck is fully CIP and water cured, YES
    - $6.0 < S_e / h = 2090/ 190 = 11.0 < 18.0$, OK
    - Core depth = 205 - 60 - 25 = 120 mm > 100 mm, OK
    - Effective length = 2090 mm < 4100 mm, OK
    - Minimum slab depth = 175 mm < 190 mm, OK
    - Overhang 990 mm > 5h = 5 x 190 = 950 mm, OK
    - $f_c' = 30$ Mpa > 28 Mpa, OK
    - Deck must be made composite with girder, YES

2. *Reinforcement Requirements.*

    - Four layers of isotropic reinforcement, $f_y \geq 400$ Mpa
    - Outer layers placed in direction of effective length.

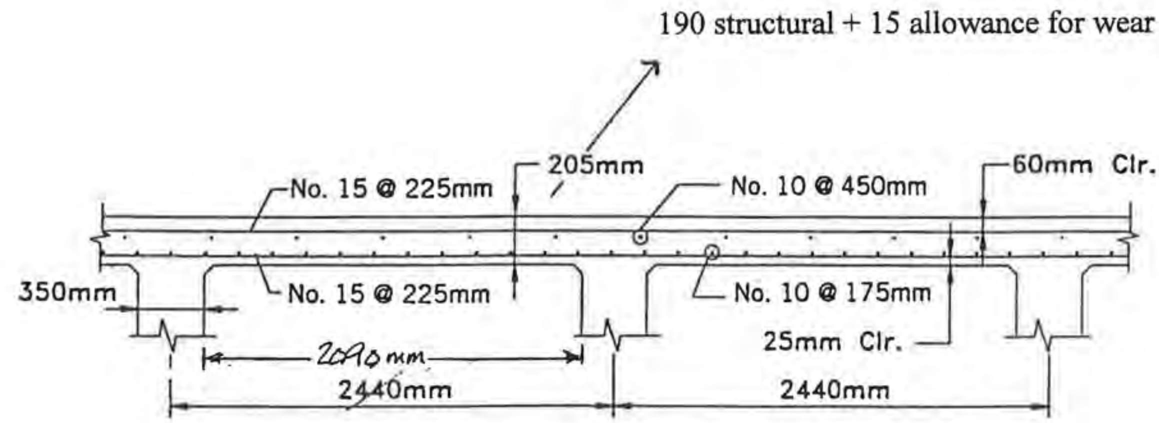


Fig. 4.29 Traditionally Designed Interior Deck Spans (6)

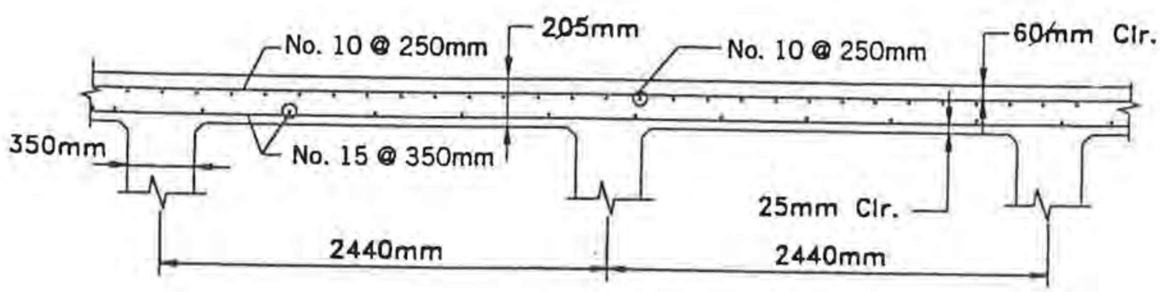


Fig. 4.30 Empirically Designed Interior Deck Spans (6)

- Bottom layers: min $A_s = 0.570$ mm^2/mm, No. 15 @ 350 mm
- Top layers: min $A_s = 0.380$ mm^2/mm, No. 10 @ 250mm
- Max spacing = 450 mm
- Straight bars only, hooks allowed, not truss bars.
- Only lap splices, no welded or mechanical splices permitted.
- Overhang designed for
  - Wheel loads using equivalent strip method if barrier discontinuous.
  - Equivalent line loads if barrier continuous.
  - Collision loads using yield line failure mechanism.

Note in Fig. 4.30 that the empirically designed slab has an isotropic mat in the top of the slab and an isotropic mat in the bottom. Also, the total reinforcing steel required in the empirical design is significantly less for the empirically designed slab (approximately 74% of that of the strip method design).

It should be noted that neither the traditional method nor the empirical method for the design of deck slabs includes the design of the deck overhang. The design loads for the deck overhang are applied to a free-body diagram of a cantilever that is independent of the deck spans. The resulting overhang design can then be incorporated into either the traditional or empirical design by anchoring the overhang reinforcement into the first deck span.

4.9 Plastic Collapse Analysis of Plates/Decks

Plastic collapse analysis or yield-line theory provides a way to estimate the load-carrying capacity of reinforced concrete slabs and ductile metal plates. Assume that on some cross section of an isotropic and ductile plate, all material through the plate thickness has yielded, with σ_y the normal stress of greatest magnitude, as shown in Fig. 4.31. The associated "fully plastic" bending moment per unit length, M_{fp}, can be calculated as indicated in Fig. 4.31.

Let us now consider a square plate simply supported on all four edges and loaded by uniform lateral pressure q as shown in Fig. 4.32 When q reaches its plastic collapse value q_c,

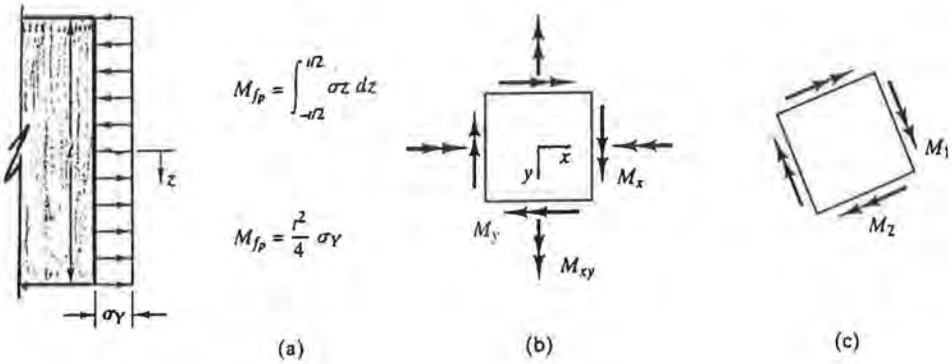


Fig. 4.31 (a) Stress distribution through the plate thickness that provides fully plastic bending moment m_{fp}. (b) Plate element in xy coordinates. (c) Principal element and principal bending moments.

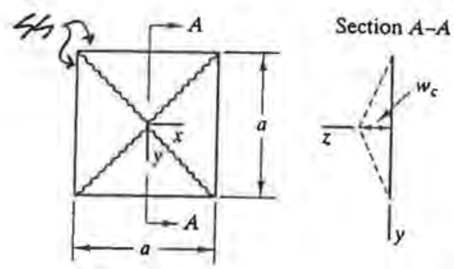


Fig. 4.32 Simply Supported Square Plate

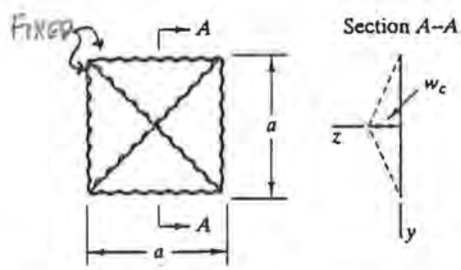


Fig. 4.33 Fixed Boundary Square Plate

then bands of yielded material called "yield lines" appear along diagonals of the plate. Due to symmetry, no shear stress is present along a diagonal. Hence $M_{xy} = 0$ along diagonals, and moment vectors parallel and normal to diagonals are principal moments M_1 and M_2. Therefore the moment vector parallel to a diagonal is $M_1 = M_{fp}$.

An upper bound on collapse load q_c can be obtained from a virtual work argument, using the assumed collapse mechanism shown in Fig. 4.32. Elastic deformation accumulated prior to collapse is not shown. Center deflection w_c, is regarded as a virtual displacement that occurs after M_{fp} has developed along yield lines. Thus each of four triangular portions of the plate rotates as a rigid body about its edge support, through the small angle $\theta = w_c /(a/2)$. Force resultant $F = q_c a^2 /4$ on a triangular portion acts at its centroid, which has displacement $w_c /3$, and so does work $F(w_c /3)$. Work absorbed by plastic moment M_{fp} is most easily calculated by projecting moment resultants on axes of rotation. For a single triangular portion, the projected moment is $M_{fp}a$, which is associated with rotation θ. For equilibrium, net virtual work must vanish. Thus summing over all four triangular portions, we obtain the upper bound solution for collapse pressure q_c.

$$4\left[\frac{q_c a^2}{4}\left(\frac{w_c}{3}\right)\right] - 4\left[\, M_{fp} a\left(\frac{w_c}{a/2}\right)\,\right] = 0 \qquad (4.44a)$$

hence

$$q_c = \frac{24 M_{fp}}{a^2} \qquad (4.44b)$$

If the square plate of Fig. 4.32 had been loaded with a P load at its center rather than the uniform load, then energy considerations would yield

$$P_c\,(w_c) - 4\left[M_{fp}a\left(\frac{w_c}{a/2}\right)\right] = 0 \tag{4.45a}$$

or,

$$P_c = 8M_{fp} \tag{4.45b}$$

If all of the edges of the plate are clamped as shown in Fig. 4.33, application of energy principles yields the following upper bound solutions,

$$q_c = \frac{48M_{fp}}{a^2} \tag{4.46}$$

$$P_c = 16M_{fp} \tag{4.47}$$

For the case of a circular plate with a uniform load which is simply supported as shown in Fig. 4.34, energy principles yield

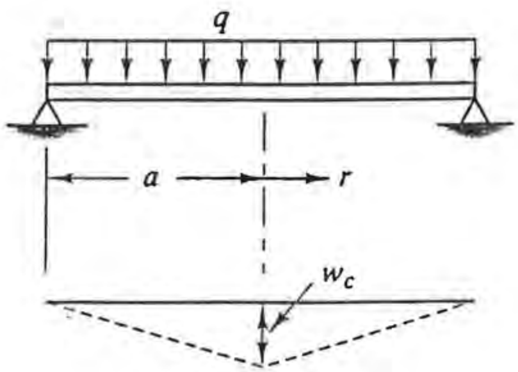
$$q_c = \frac{6M_{fp}}{a^2} \tag{4.48}$$

Or, if it is loaded with a central concentrated P load, then

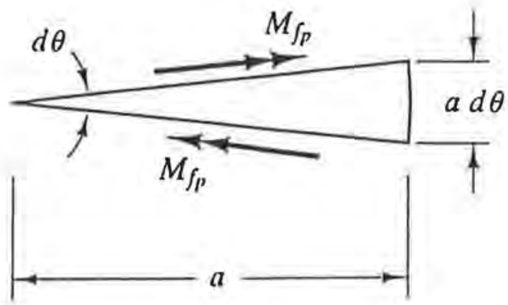
$$P_c = 2\pi M_{fp} \tag{4.49}$$

If the circular plate has clamped boundaries and a uniform load as shown in Fig. 4.35, then

$$q_c = \frac{12\ M_{fp}}{a^2} \tag{4.50}$$



a. collapse mechansim



b. bending moments on typical sector of plate

Fig. 4.34 Simply Supported Circular Plate

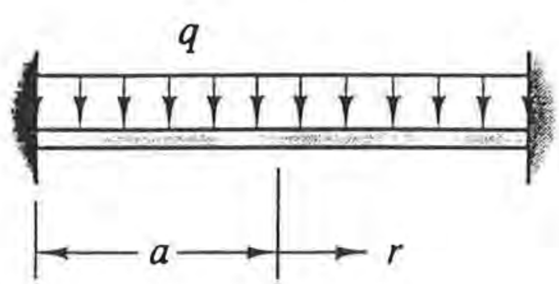


Fig. 4.35. Clamped Circular Plate.

If the uniform load is replaced with a central P load, then

$$P_c = 4\pi M_{fp} \tag{4.51}$$

In the analyses and results above, we have assumed that the material is isotropic. This assumption is approximately true for a concrete slab it is "isotropically reinforced," since the plastic condition is associated with yielding of the reinforcement bars. Even when not isotropically reinforced, we typically still treat it as an isotropic material. Note in the results above, i.e., Eqns. (4.44b), (4.46), (4.48), and (4.50), that for uniformly loaded plates

$$q_c = f\left(\frac{M_{fp}}{a^2}\right) \tag{4.52a}$$

or,

$$q_c = f\left(\frac{t^2 \sigma_y}{a^2}\right) \tag{4.52b}$$

for materials with M_{fp} stress distributions as shown in Fig. 4.31.

For concrete slabs,

$$q_c = f\left(\frac{\sigma_y t}{a^2}\right) \tag{4.53}$$

and thus the load capacity varies directly with the slab thickness, t, and inversely with the square of the span length parameter, a. Thus closer girder spacings and thicker decks would give greater ultimate load capacities for slabs under uniform load.

However, for plates under concentrated central loads, Eqns. (4.45b), (4.47), (4.49), and (4.51) reflect that the flexural load capacity is independent of the plate span length and only a function of M_{fp}, where,

$$M_{fp} = \frac{t^2}{4}\sigma_y \tag{4.54}$$

for homogenious and isotropic materials, and

$$M_{fp} = A_s f_y \left(d - \frac{a_c}{2} \right) \tag{4.55}$$

for concrete slabs. Thus for concrete slabs, thicker decks would provide greater ultimate P load capacities if the same reinforcing steel force, $A_s f_y$, is used.

If one assumes the edge conditions of a bridge deck slab are somewhere between being simply supported and clamped, and further assumes the concentrated yield line load, P_c, to be half-way between those for simply supported and clamped conditions, then,

$$P_c \approx 12\, M_{fp} \tag{4.56a}$$

$$P_c \approx 12\, A_s f_y \left(d - \frac{a_c}{2}\right) \tag{4.56b}$$

If the same reinforcing steel force, $A_s f_y$, is used,

$P_c \approx$ linear function of d

Thus, increasing the deck thickness from say 7" to 8", would result in increasing the effective slab thickness d from 5.75" to 6.75" and thus increase P_c by 19% as reflected in Table 4.6.

For bridge decks, concentrated truck wheel loads would be the governing design load, and in addition to concerns about deck flexural capacities, "punching shear" with its associated large transverse shear stresses under the wheel loads is a significant failure concern.

Table 4.6. Approximate Relative Yield-Line Collapse Loads for Various Deck Thicknesses*

Deck Thickness T (in)	Effective Depth d (in)	Moment Arm $(d - \frac{a}{2})$ (in)	$P_c/P_c^{7''**}$	$P_c/P_c^{6''**}$
6	4.75	4.25	0.81	1.00
7	5.75	5.25	1.00	1.24
8	6.75	6.25	1.19	1.47
9	7.75	7.25	1.38	1.71

*Assuming the same steel force, $A_s f_y$, is employed for all.
**See Eqn. (4.56b)

4.10 Construction Tolerances and Errors

The AASHTO requirements point out that construction tolerances become a concern for thin decks. This illustrated in Table 4.7 where it can be seen that construction errors of -1 in. on deck thickness and +0.5 in. on top bar cover would result in significant reductions in cracking moment capacities, ultimate moment capacities, and live load deflections. Also note in Table 4.7 that these errors are of greater significance for the thinner decks. Whereas the -1" error on deck thickness and +0.5" error on top bar cover are out of range of acceptance for construction tolerance (which would probably be around ± ¼"), the errors are smaller than were found by UAB researchers on many bridge decks in Birmingham (9).

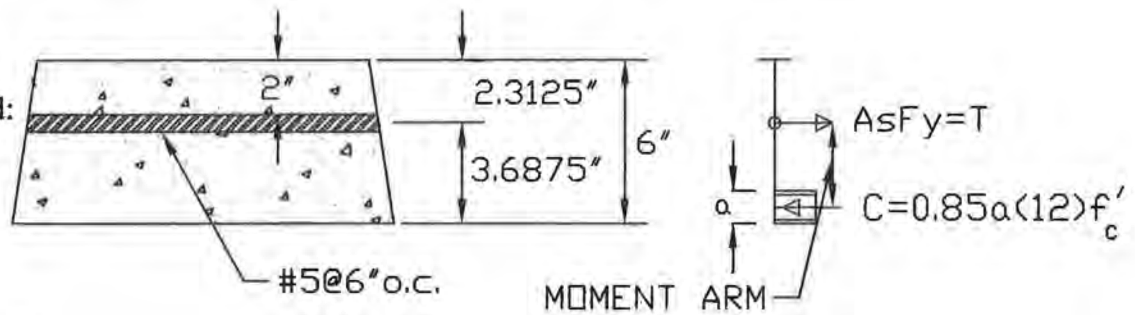
Let us examine the effect of construction tolerances and errors on the ultimate moment

Table 4.7 Effects of Construction Errors On Deck Moment Capacities and Deflections

Deck Stiffness, Strength, Deflection Parameters	Specified Deck D = 6" Cover = 2"	As-Built Deck D = 5" Cover = 2½"	As-Built Deck/ Specified Deck	Specified Deck D = 7" Cover = 2"	As-Built Deck D = 6" Cover = 2½"	As-Built Deck/ Specified Deck	Specified Deck D = 8" Cover = 2"	As-Built Deck D = 7" Cover = 2½"	As-Built Deck/ Specified Deck
$S = \frac{12D^2}{6} = 2D^2$	72 in^3	50 in^3	0.69	98 in^3	72 in^3	0.73	128 in^3	98 in^3	0.77
$M_{CR} = f_r * S = f_r \cdot (2D^2)$	72 $in^3 \cdot f_r$	50 $in^3 \cdot f_r$	0.69	98 $in^3 \cdot f_r$	72 $in^3 \cdot f_r$	0.73	128 $in^3 \cdot f_r$	98 $in^3 \cdot f_r$	0.77
$I = \frac{12D^3}{12} = D^3$	216 in^4	125 in^4	0.58	343 in^4	216 in^4	0.63	512 in^4	343 in^4	0.67
$\Delta_{LL} = \frac{k_D}{I} = \frac{k_D}{D^3}$	$\frac{k_D}{216\ in^4}$	$\frac{k_D}{125\ in^4}$	1.73	$\frac{k_D}{343\ in^4}$	$\frac{k_D}{216\ in^4}$	1.59	$\frac{k_D}{512\ in^4}$	$\frac{k_D}{343\ in^4}$	1.49
d = D - cover	4 in	2.5 in	0.63	5 in	3.5 in	0.70	6 in	4.5 in	0.75
$M_{ult} = \varphi A f_y (d - \frac{a}{2})$									
$M_{ult} = k_u (D - cover - \frac{a}{2})$	$4k_u - \frac{a}{2} k_u$	$2.5k_u - \frac{a}{2} k_u$	0.63	$5k_u - \frac{a}{2} k_u$	$3.5\ k_u - \frac{a}{2} k_u$	0.70	$6k_u - \frac{a}{2} k_u$	$4.5k_u - \frac{a}{2} k_u$	0.75

capacity of decks. We will assume as a base of comparison a 6" thick deck with #5 bars at 6" o.c. in the transverse direction with 2" of cover as shown below.

As Specified:



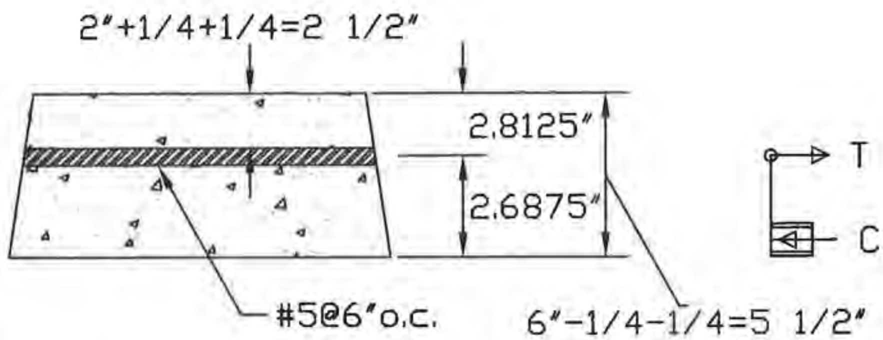
$C = T$

$$a = \frac{2A^{\#5}Fy}{.85 \ x \ 12 \ x \ f_c} = \frac{2 \ x \ 0.307 \ x \ 60}{.85 \ x \ 12 \ x \ 4} = 0.90''$$

$$Moment \ Arm = 3.6895 - \frac{0.90}{2} = 3.236''$$

Assume a ± ¼" tolerance on the rebar cover and the deck thickness, and a ± ¼" construction error (or out-of-spec) on these same parameters. Thus the worst case situation regarding reduction in moment arm and ultimate moment capacity would be as shown below.

As Built:



$C = T$

$a = 0.90''$

$$Moment \ Arm = 2.6875'' - \frac{0.90}{2} = 2.236''$$

Comparing the As-Built with the As-Specified moment arms reveals a 1.0", or

$$\frac{1.0}{3.236} \ x \ 100 = 31\%$$

reduction in moment arm and ultimate moment capacity due to the above assumed construction tolerances and errors for a 6" deck thickness.

If the deck thickness had been 7", 8", or 9", and the other parameters remain as shown above, then the reductions in moment capacity due to these same construction tolerances and errors would be as shown in Table 4.8. Note in this table the reduced effect of construction tolerances and errors on M_{ult} as deck thickness increases. This alone would be a good reason for requiring thicker decks.

Table 4.8. Effect of Construction Tolerances and Errors on M_{ult} for Different Deck Thicknesses

Deck Thickness (in)		Moment Arm (in)		% Reduction
Specified	As-Built	Specified	As-Built	Moment Arm and M_{ult}
6	5.5	3.236	2.236	31%
7	6.5	4.236	3.236	24%
8	7.5	5.236	4.236	19%
9	8.5	6.236	5.236	16%

5. PARAMETER SENSITIVITY TO DECK THICKNESS

5.1 General

The main parameters effecting and/or effected by bridge deck thickness have been discussed in Chapters 2 and 4. The purpose of this chapter is to summarize these parameters and to examine them more closely and quantitatively. More specifically we want to examine the effect of deck thickness on,

- Deck Weight and Transverse Cross-Section Properties
- Deck/Girder Longitudinal Properties (dependents on girder type/size)
- Deck Thermal and Drying Shrinkage Stresses and Cracking Propensity
- Deck Cracking, Deflection, and Vibration Characteristics
- Deck Flexural, Punching Shear, and Strength Characteristics
- Deck Influence Lines for Girder and-Slab Load Distribution Coefficients
- Deck Fatigue / Service Life
- Deck/Bridge Costs

The effects will primarily be evaluated from theoretical viewpoints rather than that of design codes. Simplified code equations are used for convenience and economy of time in the design process, and hopefully these equations are accurate and/or slightly conservative. However this is not always the case and actual bridge/deck behaviors and stresses determine the actual damage and deterioration done by loadings, and these are best predicted by theoretical considerations.

5.2 Deck Weight and Transverse Cross-Section Properties

Equations used in calculating the deck unit weight (q), uncracked section modulus (S), uncracked moment of inertia (I_g), cracked moment of inertia (I_{cr}), and torsional stiffness (J) are given below. These equations were used to compute the parameters indicated for deck thicknesses of 6'', 7'', 8'', 9'' and the results are shown in Table 5.1. Also shown in Table 5.1 are the parameter values normalized to those for the 6''deck thickness. Fig. 5.1 shows plots of the normalized parameters to better illustrate the trends and for convenience of comparison.

$$\textit{Deck Unit Weight: } q = \frac{d''}{12} \cdot 150 lb/ft = 12.5 \cdot d'' \qquad psf$$

$$\textit{Uncracked Moment of Inertia: } I_g = \frac{bd^3}{12} = \frac{12'' d^3}{12} \qquad in^4$$

$$\textit{Section Modulus: } S = \frac{I_g}{C} \qquad in^3$$

$$\textit{Torsional Stiffness: } J = \frac{bd^3}{3} = \frac{(1)d^3}{3} \qquad in^4$$

$$\textit{Cracked Moment of Inertia: } I_{cr} = \left(\frac{bd^3}{12} + aD^2\right)_{con} + \left(\frac{bd^3}{12} + aD^2\right)_{steel} \times \frac{E_{con}}{E_{steel}} \qquad in^4$$

Note in the above equations that deck unit weight increases as the first power of d, whereas the section modulus (and thus reductions in service load transverse stresses) increases with d^2, the moment of inertia (thus reductions in the service load transverse deflections), and the torsional stiffness increases with d^3. Note in Fig. 5.1 the significant increases in the deck section modulus and stiffness parameters with increasing deck thickness while, there is only a small increase (compared to the other parameters) in deck unit weight.

Since the ALDOT is considering an increase in minimum deck thickness from 7'' to 8'', Table 5.2 gives the values of the parameters for 7'' and 8'' deck thickness when normalize w.r.t. the 7'' deck thickness. Again, note the significant increases in section modulus and stiffness when increasing the deck thickness from 7'' to 8''.

Table 5.1 Deck Cross Section Parameters and Parameters Normalized w.r.t. 6'' Deck Thickness

D	I_G (in^4)	S (in^3)	I_{CR} (in^4)	J (in^4)	q (psf)	$I_G/I_G^{6''}$	$S/S^{6''}$	$I_{CR}/I_{CR}^{6''}$	$J/J^{6''}$	$q/q^{6''}$
6''	216	72	85	72	75	1	1	1	1	1
7''	343	98	113	114	88	1.6	1.4	1.3	1.6	1.2
8''	512	128	142	171	100	2.4	1.8	1.7	2.4	1.3
9''	729	162	170	243	113	3.4	2.3	2.0	3.4	1.5

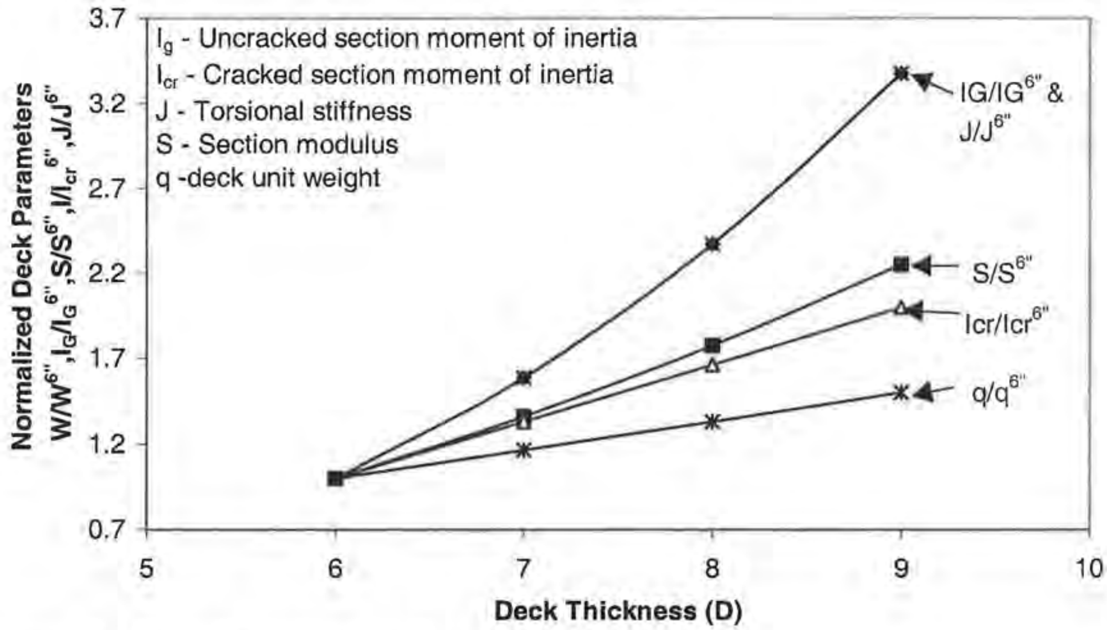


Figure 5.1 Deck Cross Section Properties

Table 5.2 Deck Cross Section Parameters Normalized w.r.t. 7'' Deck Thickness

D	$I_G/I_G^{7''}$	$S/S^{7''}$	$I_{CR}/I_{CR}^{7''}$	$J/J^{7''}$	$q/q^{7''}$
7''	1	1	1	1	1
8''	1.49	1.31	1.25	1.49	1.14

5.3 Deck/Girder Longitudinal Properties

For deck longitudinal properties three different girder types were selected to show the effect of deck thickness for different girders. The girders selected were AASHTO Type II with a span of 45 ft, AASHTO Type VI with a span of 125 ft, and W36x170 with a span of 70 ft. In each case a girder spacing of 8 ft and a b_{eff} of 96'' was used. Equations used in calculating the deck/girder unit weight (q), uncracked moment of inertia (I_g), uncracked section modulus (S), maximum deck longitudinal stress (σ), deck-superstructure fundamental frequency (f) and period (T) and maximum live load deflection (Δ_{LL}) are given below. These equations were used to compute the parameters indicated for deck thicknesses of 6'', 7'', 8'', 9'' and the results are shown in Tables 5.3, 5.5, and 5.7. Also shown in Tables 5.3, 5.5, and 5.7 are the parameter values normalized to those for the 6''deck thickness. Figs. 5.2, 5.3, and 5.4 show plots of the normalized parameters to better illustrate the trends and for convenience of comparison.

Deck/Girder Unit Weight: $q = \frac{b_{eff}\, d}{12} \cdot 150 + q_{girder} = 100 \cdot d + q_{girder}$ psf

Uncracked Moment of Inertia: $I_g = \left(\frac{bd^3}{12} + aD^2 \right)_{slab} + \left(I + aD^2 \right)_{girder}$ in^4

Section modulus: $S_t = \frac{I_g}{y_t}$ in^3

Live load deflection: $\Delta_{LL} = 0.001 \frac{PL^3}{EI}$ *in*

Where P = *16000 lbs*

Deck Stress: $\sigma = \frac{M}{S}$ *psi*

Using $M = \frac{PL}{4}$ *where L = span length*

M = 2160000 lb-in (AASHTO Type II)

M = 6000000 lb-in (AASHTO Type VI)

M = 3360000 lb-in (W36x170 Steel Girder)

Frequency: $f = \sqrt{\frac{EI\lambda^4}{m}}$ *Hz*

Period: $T = \frac{1}{f}$ *sec*

Note in Figs. 5.2, 5.3, and 5.4 the significant increases (for every girder type) in the deck section modulus, uncracked moment of interia, and fundamental frequency while there was a significant decrease in live load deflection, maximum longitudinal deck stress, and period. Note also in Figs. 5.2, 5.3, and 5.4 a significant increase in deck/girder unit weight for the given b_{eff}. Also note that the AASHTO Type II girder had the largest increases in section modulus, uncracked moment of interia, and largest decreases in live load deflection and maximum longitudinal deck stress as deck thickness increases. While the W36x170 steel girder had the largest increases in deck/girder unit weight, frequency and largest decreases in period with increasing deck thickness.

Again, since the ALDOT is considering an increase in minimum deck thickness from 7'' to 8'', Tables 5.4, 5.6, and 5.8 gives the values of the parameters for 7'' and 8'' deck thickness when normalize w.r.t. the 7'' thickness. Again, note the significant increases in section modulus, uncracked moment of interia, frequency, and deck/girder unit weight, and decreases in maximum longitudinal stress, period, and live load deflections when increasing the deck thickness from 7'' to 8''.

Table 5.3 Longitudinal Deck/Girder Parameters and Parameters Normalized w.r.t. 6'' Deck Thickness for AASHTO Type II Bridge Girders

D	6''	7''	8''	9''
q(psf)	984	1084	1184	1284
$I_G(in^4)$	173452	187180	200681	214179
$S(in^3)$	14398	15742	16944	18024
σ(psi)	150.021	137.213	127.479	119.840
f(Hz)	10.421	12.246	14.097	15.973
T(sec)	0.096	0.082	0.071	0.063
$\Delta(10^{-3}in)$	4.032	3.728	3.488	3.264
$q/q^{6''}$	1	1.102	1.203	1.305
$I_G/I_G^{6''}$	1	1.079	1.157	1.235
$S/S^{6''}$	1	1.093	1.177	1.252
$\sigma/\sigma^{6''}$	1	0.915	0.850	0.799
f/f6''	1	1.175	1.353	1.533
$T/T^{6''}$	1	0.851	0.739	0.653
$\Delta/\Delta^{6''}$	1	0.926	0.864	0.810

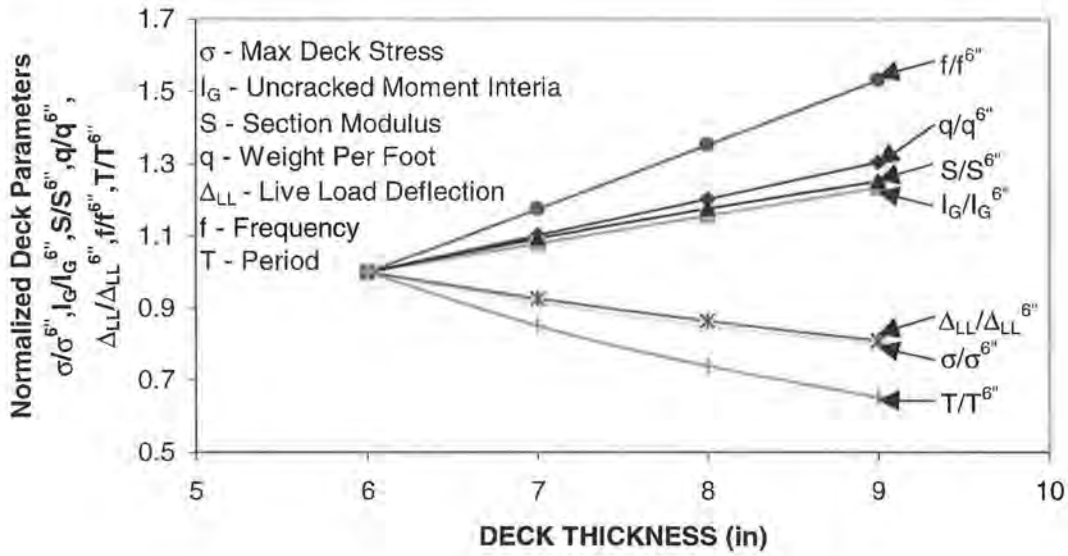


Figure 5.2 Longitudinal Deck/Girder Parameters-AASHTO Type II Bridge Girders

Table 5.4 Longitudinal Deck/Girder Parameters Normalized w.r.t. 7''DeckThickness for AASHTO Type II Bridge Girders

D	$q/q^{7''}$	$I_G/I_G^{7''}$	$S/S^{7''}$	$\sigma/\sigma^{7''}$	$f/f^{7''}$	$T/T^{7''}$	$\Delta/\Delta^{7''}$
7''	1	1	1	1	1	1	1
8''	1.092	1.072	1.0763	0.922	1.151	0.869	0.936

Table 5.5 Longitudinal Deck/Girder Parameters and Parameters Normalized w.r.t. 6'' Deck Thickness for AASHTO Type VI Bridge Girders

D	6''	7''	8''	9''
q(psf)	1730	1830	1930	2030
$I_G(in^4)$	1296234	1371139	1443318	1513353
$S(in^3)$	45921	49575	53065	56395
σ(psi)	130.659	121.029	113.068	106.392
f(Hz)	9.778	11.493	13.200	14.924
T(sec)	0.102	0.087	0.076	0.067
$\Delta(10^{-3}in)$	11.552	10.928	10.384	9.904
$q/q^{6''}$	1	1.058	1.116	1.173
$I_G/I_G^{6''}$	1	1.056	1.114	1.168
$S/S^{6''}$	1	1.080	1.156	1.228
$\sigma/\sigma^{6''}$	1	0.93	0.87	0.81
f/f6''	1	1.175	1.350	1.526
$T/T^{6''}$	1	0.851	.741	0.655
$\Delta/\Delta^{6''}$	1	0.95	0.90	0.86

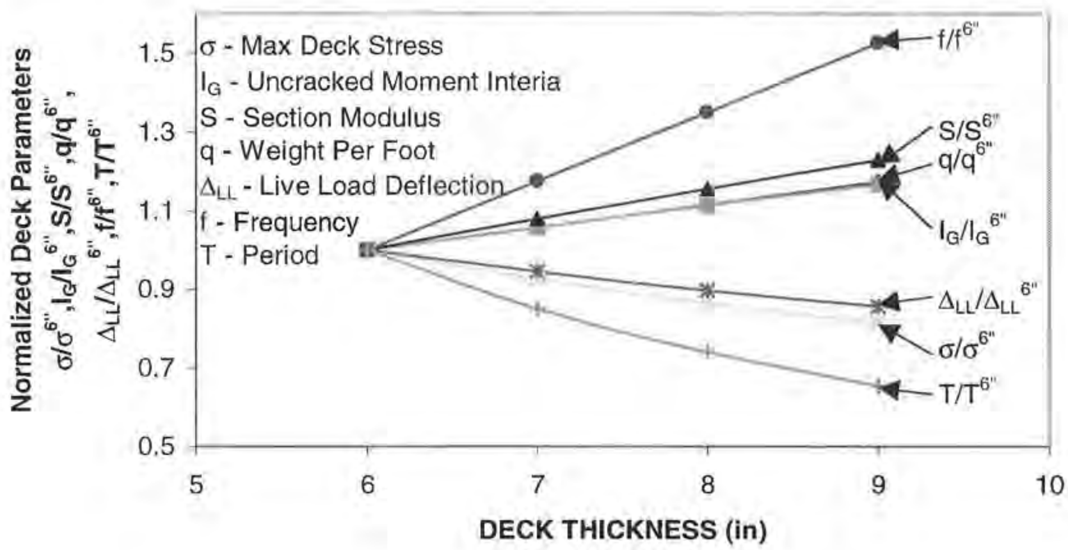


Figure 5.3 Longitudinal Deck/Girder Parameters-AASHTO Type VI Bridge Girders

Table 5.6 Longitudinal Deck/Girder Parameters Normalized w.r.t. 7'' Deck Thickness for AASHTO Type VI Bridge Girders

D	$q/q^{7''}$	$I_G/I_G^{7''}$	$S/S^{7''}$	$\sigma/\sigma^{7''}$	$f/f^{7''}$	$T/T^{7''}$	$\Delta/\Delta^{7''}$
7''	1	1	1	1	1	1	1
8''	1.055	1.053	1.070	0.934	1.149	0.871	0.950

Table 5.7 Longitudinal Deck/Girder Parameters and Parameters Normalized w.r.t. 6'' Deck Thickness for W36x170 Girders

D	6''	7''	8''	9''
q(psf)	770	870	970	1070
$I_G(in^4)$	223233	238916	254264	269524
$S(in^3)$	17317	18821	20161	21361
σ(psi)	194.029	178.524	166.658	157.296
f(Hz)	30.972	36.570	42.245	47.970
T(sec)	0.032	0.027	0.024	0.021
$\Delta(10^{-3}in)$	11.784	11.011	10.346	9.76
$q/q^{6''}$	1	1.130	1.260	1.390
$I_G/I_G^{6''}$	1	1.070	1.139	1.207
$S/S^{6''}$	1	1.087	1.164	1.234
$\sigma/\sigma^{6''}$	1	0.920	0.859	0.810
f/f6''	1	1.18	1.364	1.607
$T/T^{6''}$	1	0.847	0.733	0.622
$\Delta/\Delta^{6''}$	1	0.934	0.878	0.828

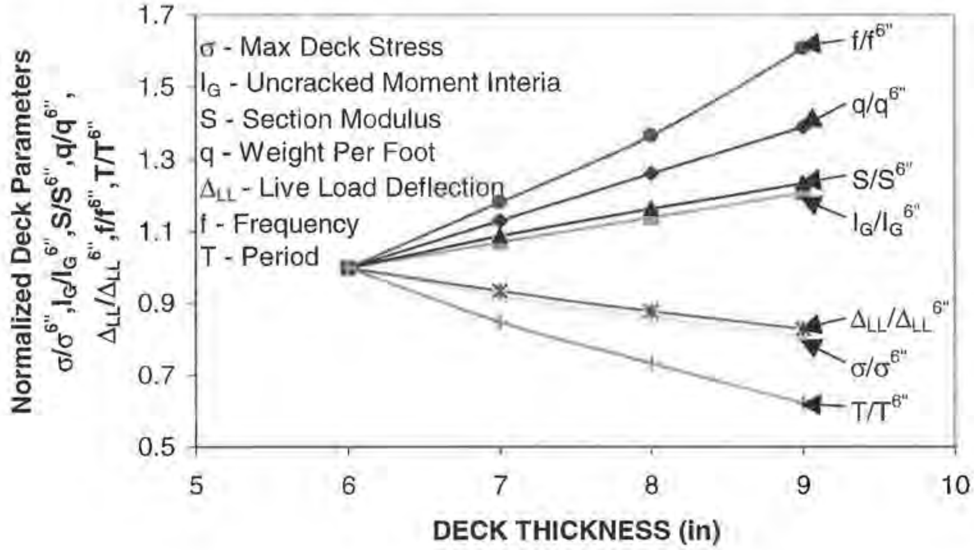


Figure 5.4 Longitudinal Deck/Girder Parameters-W36x170 Bridge Girders

Table 5.8 Longitudinal Deck/Girder Parameters Normalized w.r.t. 7''Deck Thickness for W36x170 Bridge Girders

D	$q/q^{7''}$	$I_G/I_G^{7''}$	$S/S^{7''}$	$\sigma/\sigma^{6''}$	$f/f^{7''}$	$T/T^{7''}$	$\Delta/\Delta^{7''}$
7''	1	1	1	1	1	1	1
8''	1.115	1.064	1.071	0.934	1.155	0.866	0.940

5.4 Deck Thermal and Drying Shrinkage Stresses and Cracking Propensity

Assuming full restraint and a linear temperature variation with depth, deck thermal stresses in the transverse direction will be as shown in Fig. 5.5 and thus are independent of deck thickness. For nonlinear temperature distributions, more complex stress distributions will be developed, but they too will be essentially independent of deck thickness.

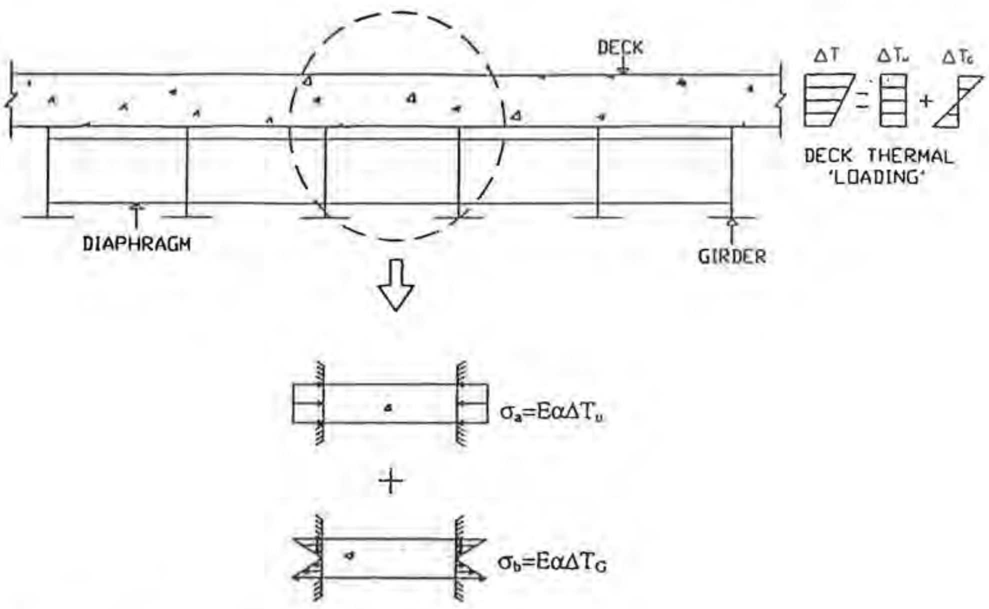


Figure 5.5 Deck Thermal Loading and Stresses

In the longitudinal directions the properties and contributions of the longitudinal girder must be brought in; but as shown in Chapter 2, the thermal stresses in the longitudinal direction are also essentially independent of the deck thickness.

Drying shrinkage stresses can be determined in the same manner as thermal stresses by converting drying shrinkage strain (which is known or estimated) to an equivalent temperature change, i.e.,

$$\Delta T_{eq} = -\frac{\varepsilon_{sh}}{\alpha}$$

Thus, the equations and observations made above for thermal stresses are equally applicable for drying shrinkage stresses. Since deck thermal and drying shrinkage stresses are essentially independent of deck thickness, so to will be the thermal and drying shrinkage cracking characteristics. These are all reflected in Fig. 5.6.

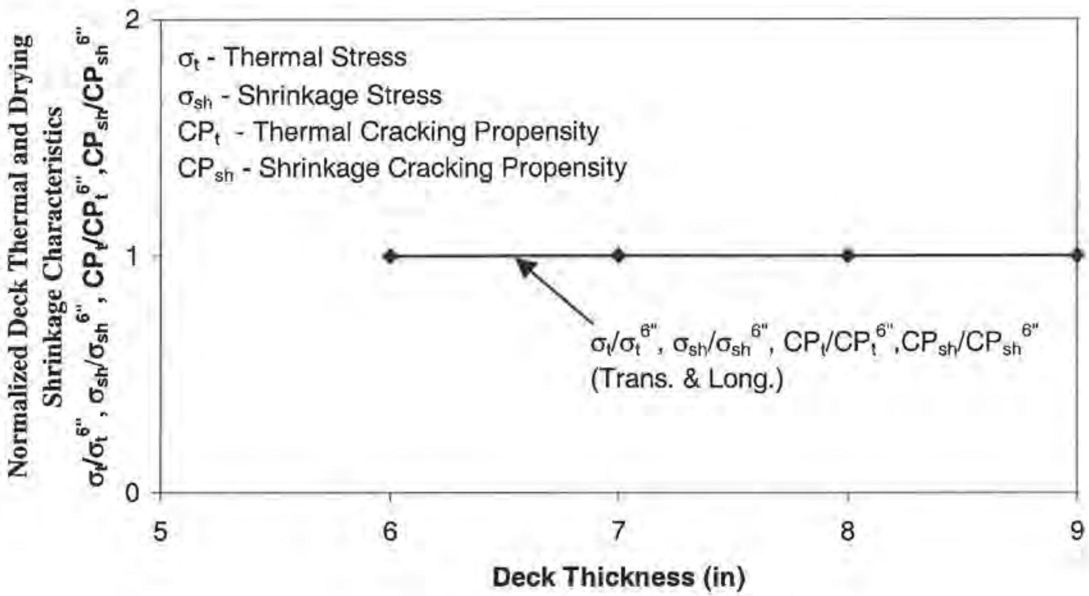


Figure 5.6 Normalized Deck Thermal and Drying Shrinkage Characteristics

5.5 Deck Cracking, Deflection, and Vibration Characteristics

Equations used in calculating the cracking propensity (CP), crack width (CW), live load deflection (Δ_{LL}), fundamental natural frequency (f) and period (T) are given below. These equations were used to compute the parameters indicated for deck thicknesses of 6'',7'', 8'', 9'' and the results are shown in Table 5.9. Also shown in Table 5.9 are the parameter values normalized to those for the 6''deck thickness. Fig. 5.7 shows plots of the normalized parameters to better illustrate the trends and for convenience of comparison.

Cracking Propensity: $CP = \dfrac{1}{d^2}$ $\quad in^{-2}$

Crack width: $CW = \dfrac{M(1')}{EI} \cdot \dfrac{d}{2}$ $\quad in$

Live load deflection: $\Delta_{LL} = 0.001\dfrac{PL^3}{EI}$ $\quad in$

where P = 16000 lb

$$Frequency:\ f = \frac{\pi d}{a^2}\sqrt{\frac{E}{12(1-\nu^2)\rho}} \qquad Hz$$

$$Where\ \nu = 0.2,\ a = 8ft,\ \rho = 4.6584 slug/ft^3$$

$$Period:\ T = \frac{1}{f} \qquad sec$$

Note in the above equations that cracking propensity and crack width decrease with the inverse of d^2, whereas the live load deflection decreases with the inverse of d^3, and the frequency increases as the first power of d. Note in Fig. 5.7 the significant decreases in the deck cracking parameters, deflection, and period of vibration, and the increase in deck frequency with increase in deck thickness

Again, since the ALDOT is considering an increase in minimum deck thickness from 7'' to 8'', Table 5.10 gives the values of the parameters for 7'' and 8'' deck thickness when normalize w.r.t. the 7'' deck thickness. Again, note the significant decreases in the deck cracking parameters, deflection, and period and the increase in the deck frequency when increasing the deck thickness from 7'' to 8''.

Table 5.9 Deck Cracking, Deflection, and Vibration Characteristics Normalized w.r.t. 6'' Deck Thickness

D	CP (in^{-2})	CW (in)	Δ_{LL} (in)	F (Hz)	T (sec)	$CP/CP^{6''}$	$CW/CW^{6''}$	$\Delta_{LL}/\Delta_{LL}^{6''}$	$f/f^{6''}$	$T/T^{6''}$
6''	0.028	3.9E-06	0.462	0.530	1.89	1	1	1	1	1
7''	0.020	2.8E-06	0.015	0.619	1.62	.73	0.735	0.630	1.167	0.857
8''	0.016	2.2E-06	0.010	0.707	1.41	.56	0.563	0.422	1.333	0.750
9''	0.012	1.7E-06	0.007	0.795	1.26	.44	0.444	0.296	1.500	0.667

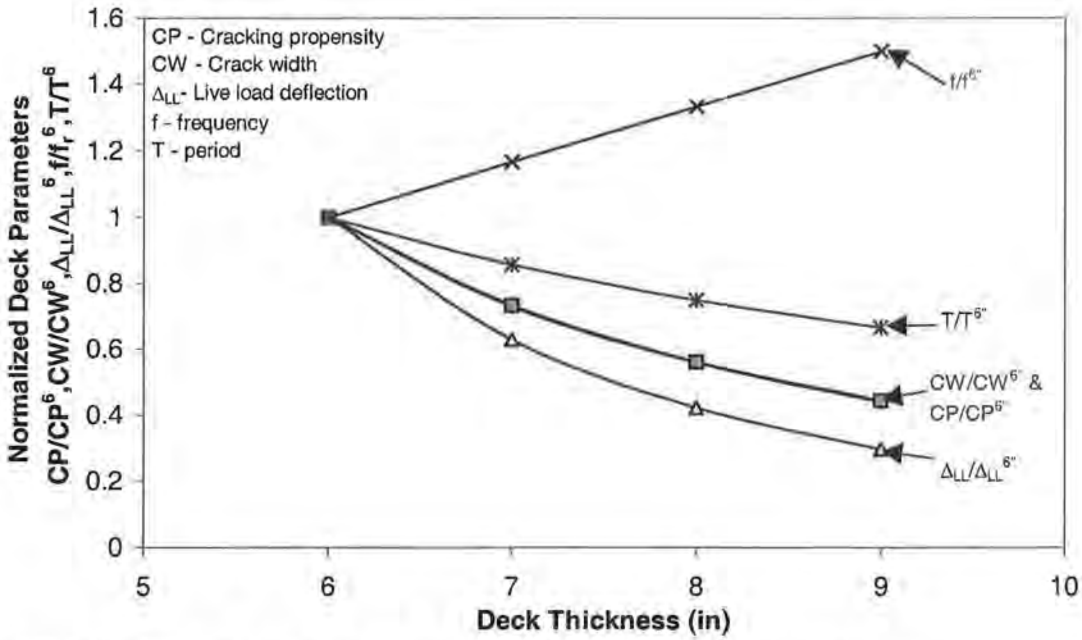


Figure 5.7 Deck Cracking, Deflection, and Vibration Characteristics

Table 5.10 Deck Cracking, Deflection, and Vibration Characteristics Normalized w.r.t. 7'' Deck Thickness

D	$CP/CP^{7''}$	$CW/CW^{7''}$	$\Delta_{LL}/\Delta_{LL}^{7''}$	$F/F^{7''}$	$T/T^{7''}$
7''	1	1	1	1	1
8''	0.8	0.7656	0.66992	1.143	0.889

5.6 Deck Flexural, Punching Shear, and Strength Characteristics

Equations used in calculating the deck stress (σ), cracking moment (M_{cr}), ultimate moment capacity (M_{ult}), yield line failure load ($P_{yl.}$), arch action force in deck (F_{arch}), and punching shear force capacity (F_{PS}) are given below. These equations were used to compute the parameters indicated for deck thicknesses of 6'', 7'', 8'', 9'' and the results are shown in Table 5.11. Also shown in Table 5.11 are the parameter values normalized to those for the 6''deck thickness. Fig. 5.8 shows plots of the normalized parameters to better illustrate the trends and for convenience of comparison.

$$Deck\ Stress:\ \sigma = \frac{M}{S} \qquad psi$$

where M = 384000 lb-in

$$Cracking\ Moment:\ M_{cr} = f_r' \cdot S \qquad k\text{-}ft$$

$$Ultimate\ Moment:\ M_{ult} = f_y \cdot (d - \frac{a}{2}) \qquad k\text{-}ft$$

$$Yield\ Line\ Failure\ Load:\ P_{yl.} = 12 A_S f_y \left(d - \frac{a_c}{2} \right) \qquad k$$

$$Arching\ Action\ Force:\ F_{arch} = \frac{P}{2} \sqrt{1 + \left(\frac{S/2}{D-2} \right)^2} \qquad k$$

where P = 16000 lb

$$Punching\ Shear:\ F_{PS} = f_v \cdot A_{shear} \qquad k$$

Note in the above equations that cracking moment increases with d^2, the ultimate moment and the yield line failure load increases with $\left(d - \frac{a}{2} \right)$, while the arching action force decreases (which is good) as the first power of (D-2), and the maximum deck stress decreases with the inverse of d^2. Note in Fig. 5.8 the significant increases in the cracking moment (by 225%), punching shear (by 185%), ultimate moment and yield line failure load by (171%), while there was a decrease in the deck arching action force (by 171%) and maximum deck stress (by 227%) when increasing the deck thickness from 6'' to 9''.

Again, since the ALDOT is considering an increases in minimum deck thickness from 7'' to 8'', Table 5.12 gives the values of the parameters for 7'' and 8'' deck thicknesses when normalize w.r.t. the 7'' thickness. Again, note the significant increases in cracking moment, punching shear, ultimate moment and yield line failure load by, while there was a decrease in the deck arching action force and maximum deck stress when increasing the deck thickness from 7'' to 8''.

Table 5.11 Deck Strength Parameters and Parameters Normalized w.r.t 6'' Deck Thickness

D	σ (psi)	M_{cr} (ft-k)	M_{ult} (ft-k)	$P_{yl.}$ (k)	F_{arch} (k)	F_{PS} (k)	$\sigma/\sigma^{6''}$	$M_{CR}/M_{CR}^{6''}$	$M_{ult}/M_{ult}^{6''}$	$P_{yl.}/P_{yl.}^{6''}$	$F_{arch}/F_{arch}^{6''}$	$F_{PS}/F_{PS}^{6''}$
6''	5333	2.84	21.3	612	97.6	96	1	1	1	1	1	1
7''	3918	3.87	26.3	756	76.8	121	0.75	1.36	1.24	1.23	0.80	1.26
8''	3000	5.06	31.3	900	64.0	149	0.56	1.78	1.47	1.47	0.67	1.55
9''	2370	6.40	36.3	1044	56.0	178	0.44	2.25	1.71	1.71	.58	1.85

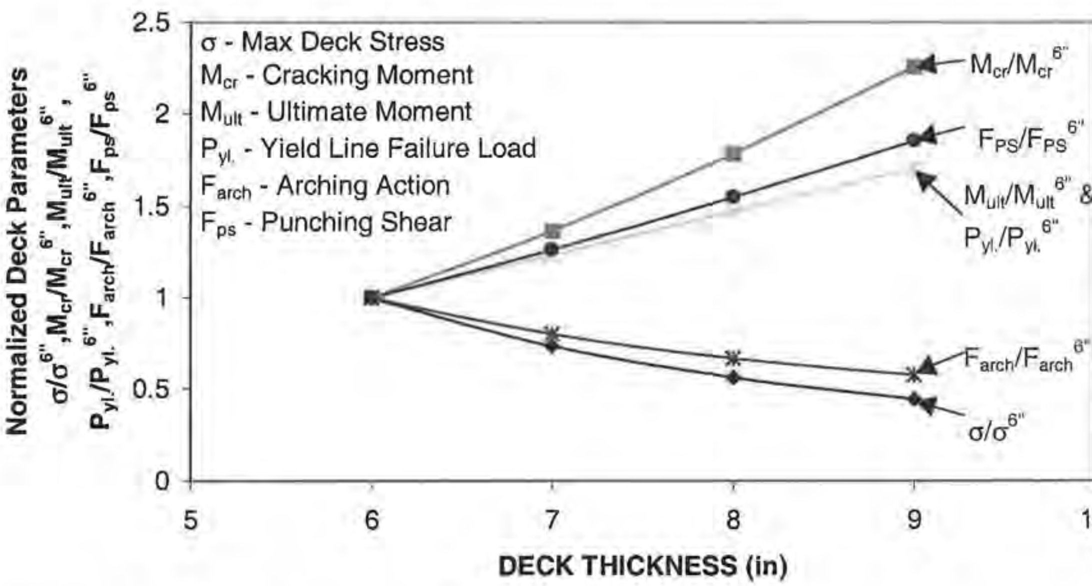


Figure 5.8 Deck Strength Parameters

Table 5.12 Deck Strength Parameters Normalized w.r.t. 7'' Deck Thickness

D	$\sigma/\sigma^{7''}$	$M_{CR}/M_{CR}^{7''}$	$M_{ULT}/M_{ULT}^{7''}$	$P_{YL.}/P_{YL.}^{7''}$	$F_{ARCH}/F_{ARCH}^{7''}$	$F_{PS}/F_{PS}^{7''}$
7''	1	1	1	1	1	1
8''	0.77	1.31	1.19	1.19	0.84	1.23

5.7 Influence Lines for Girder-and-Slab Load Distribution Coefficients

For influence lines for girder-and-slab load distribution coefficients, three different girder types were selected to show the effect of deck thickness on load distribution for different girders.

The girders selected were again, AASHTO Type II with a span of 45 ft, AASHTO Type VI with a span of 125 ft, and W36x170 with a span of 70 ft. In each case a girder spacing of 8 ft and a b_{eff} of 96'' was used. For each of these girders influence line were constructed for loads placed immediately above the edge girder, immediately above the girder next to edge girder and immediately above the centerline girder. The values for the influence lines were taken from the charts shown in Fig. 4.13(a, b, c). These charts are read by obtaining nondimensional parameters, which relate the various stiffnesses of the structure. The first is f, the flexural stiffness ratio that is equal to: $f = 0.01\frac{d^3}{l^3}\times\frac{L^4}{I}$

Thus, it can be seen that f increases with d^3. The second is r, the rotational stiffness ratio that is equal to:

$$r = 25\frac{l}{d^3}\times\frac{C}{L^2}$$

$$\begin{aligned} \text{where } C &= J_{girder} + (l\times J_{slab}) \\ L &= span\ \ length \\ l &= girder\ \ spacing \end{aligned}$$

So, it can be seen that r decreases with $\frac{C}{d^3}$. With these two parameter a value of J can be taken from the charts shown in Fig. 4.17 (a,b,c). For all of the cases it can be seen that the maximum value of J decreased with an increase in deck thickness. This means better load and moment distribution across the deck and support girders. The largest reduction of 145% in J occurs with the W36 x 170 steel girder which is more flexible than the AASHTO girders.

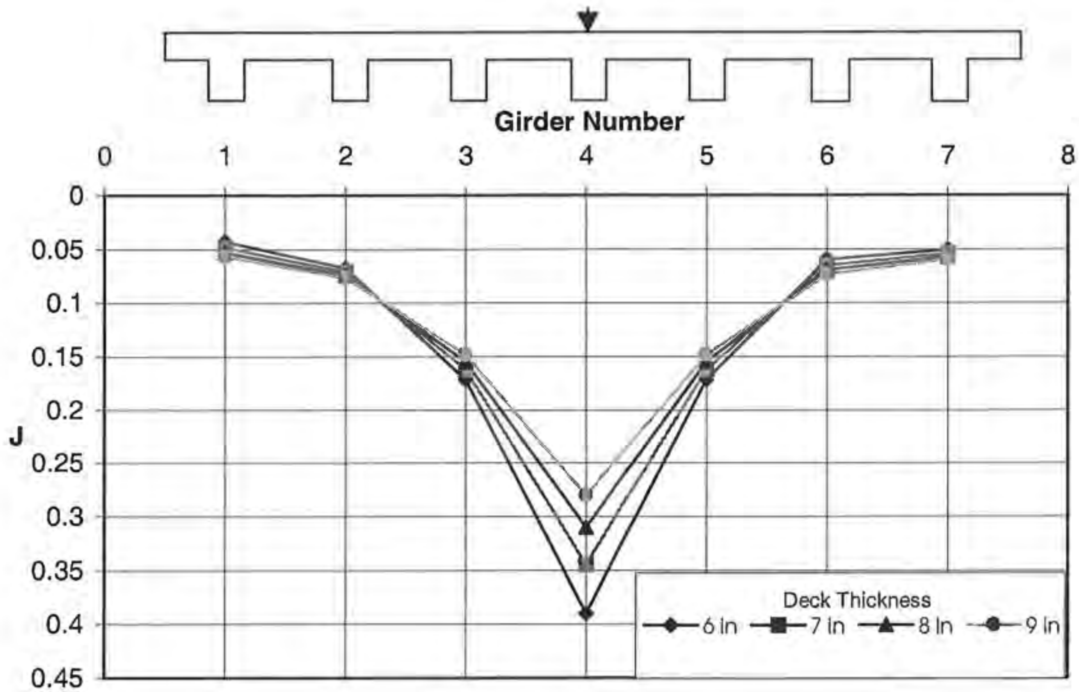


Figure 5.9 AASHTO Type II Load Placed Above Middle Girder

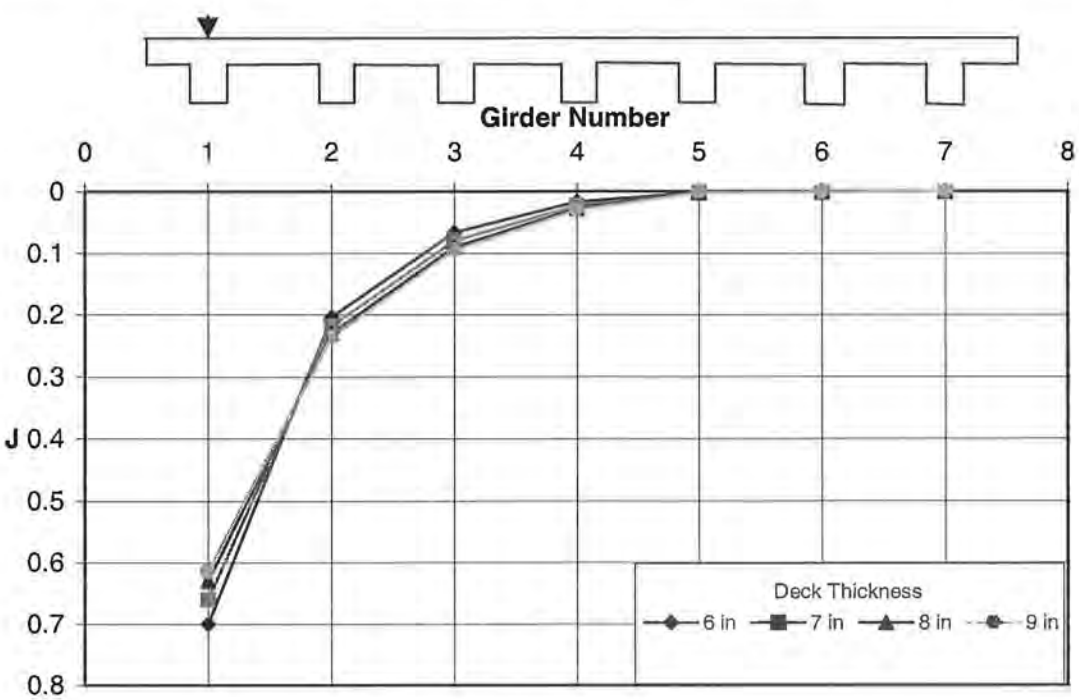


Figure 5.10 AASHTO TYPE II Load Placed Above Edge Girder

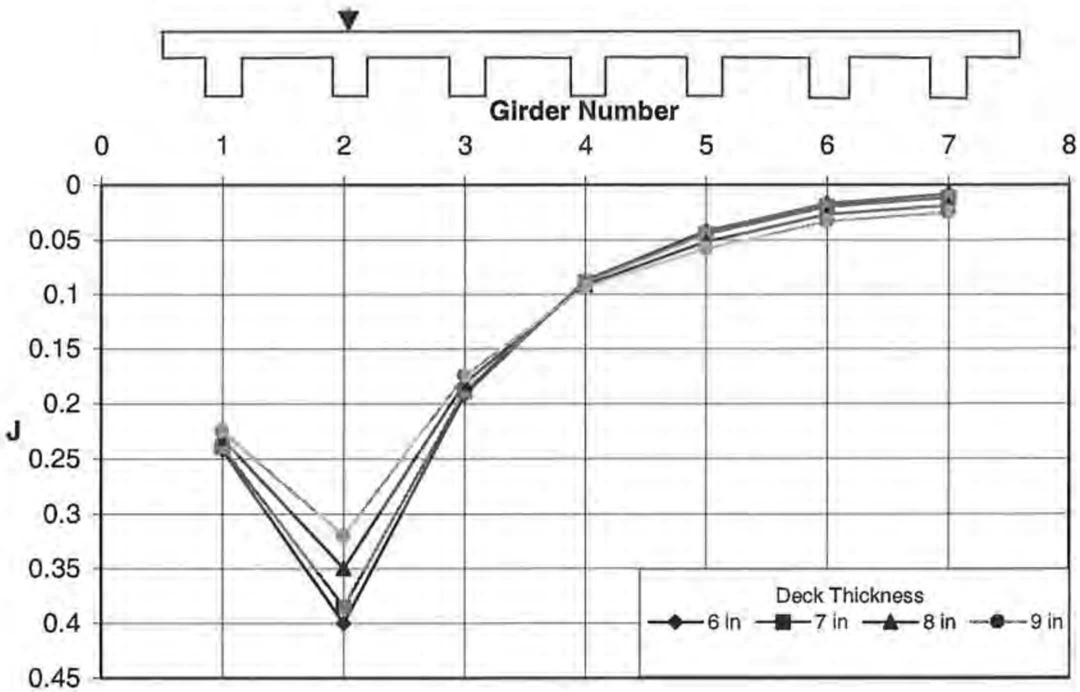


Figure 5.11 AASHTO TYPE II Load Placed Above Girder Next to Edge

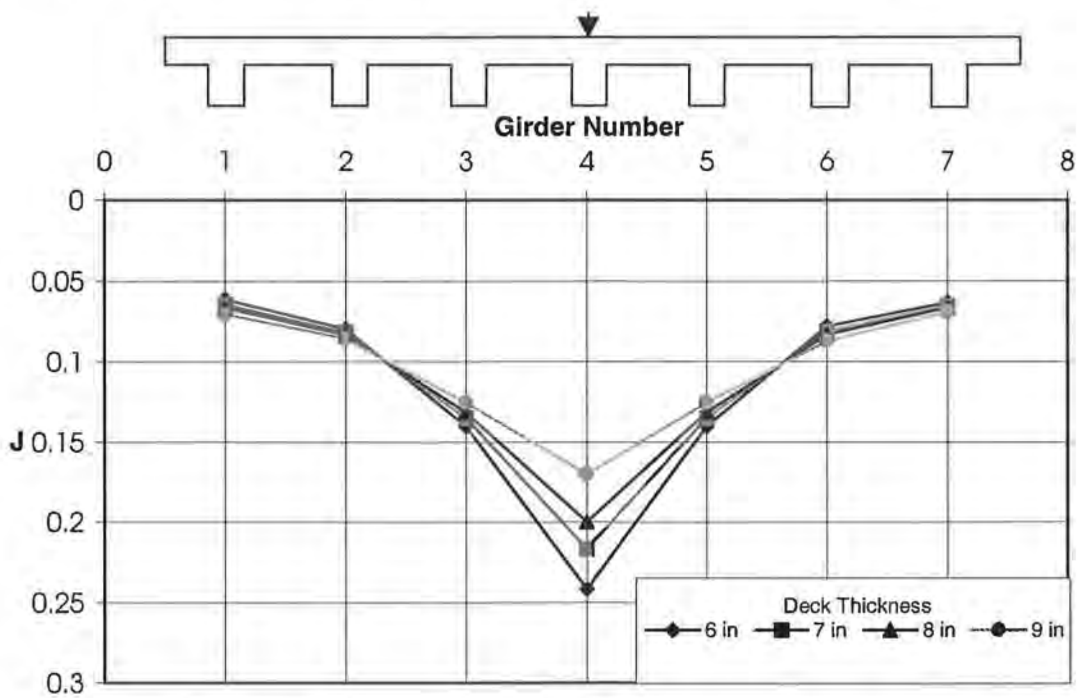


Figure 12. AASHTO Type VI Load Placed Above Middle Girder

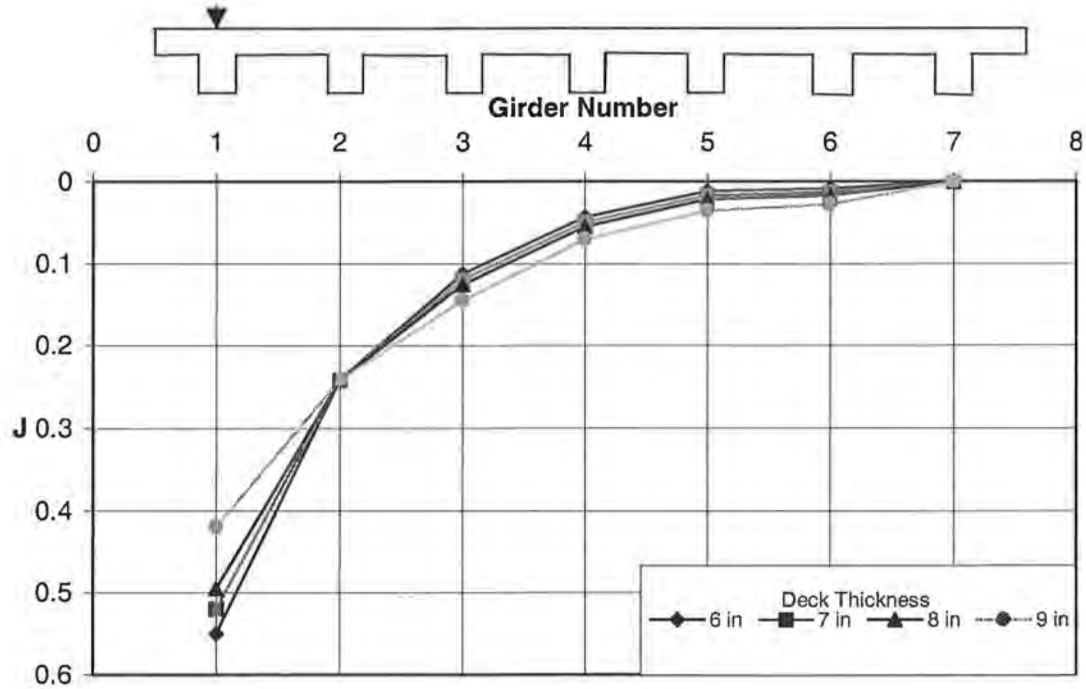


Figure 5.13 AASHTO TYPE VI Load Placed Above Edge Girder

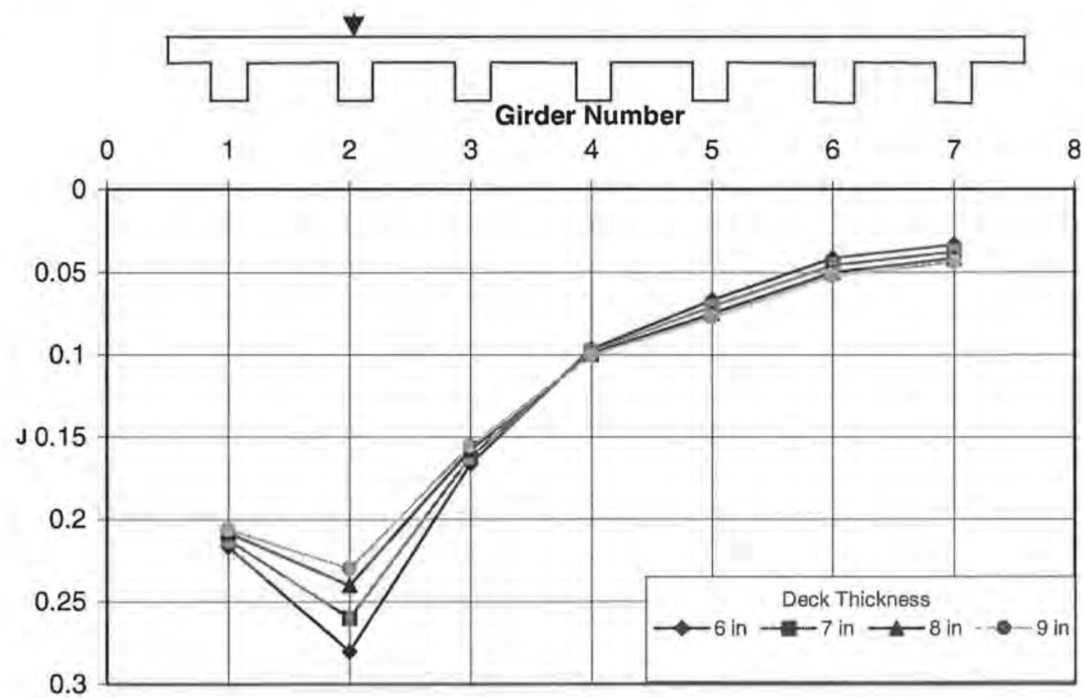


Figure 5.14 AASHTO TYPE VI Load Placed Above Girder Next to Edge

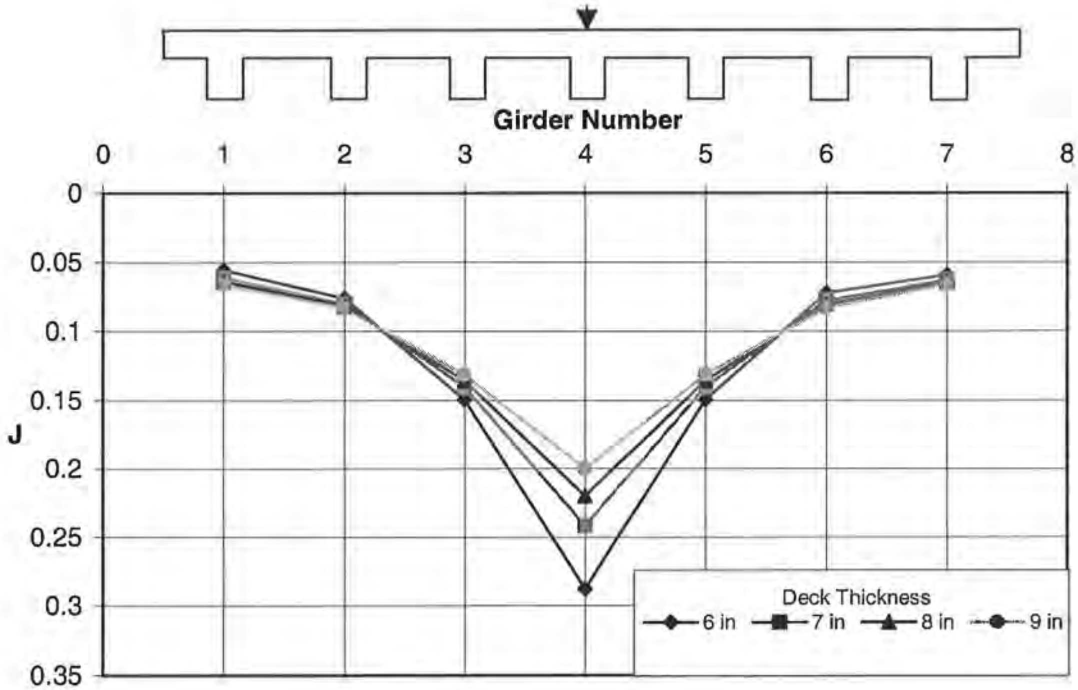


Figure 5.15 W36x170 Load Placed Above Middle Girder

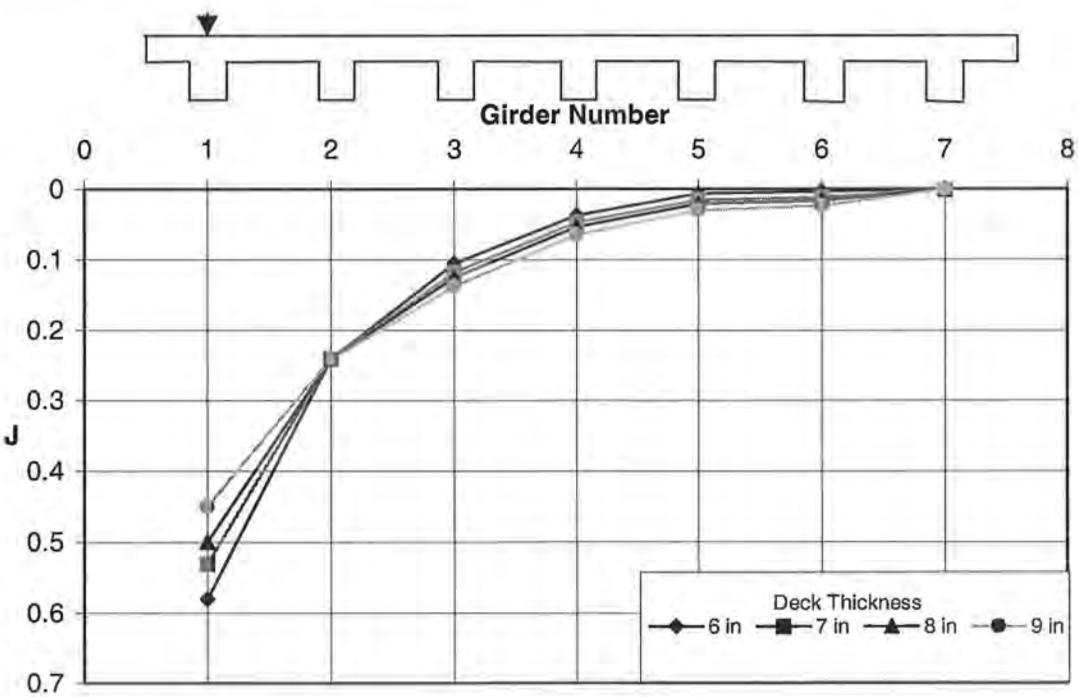


Figure 5.16 W36x170 Load Placed Above Edge Girder

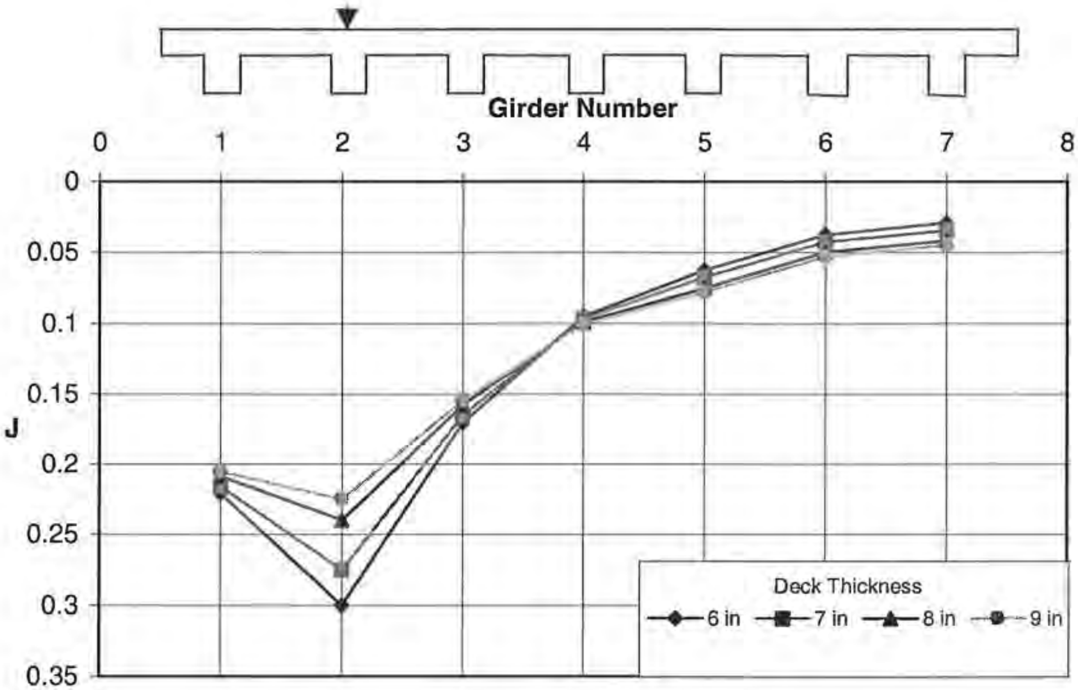


Figure 5.17 W36x170 Load Placed Above Girder Next to Edge

5.8. Deck Fatigue/Service Life

As indicated in Chapter 2, ACI Committee 215 concludes that the fatigue strength of concrete for the life of 10 million cycles of load and a probability of failure of 50% regardless of whether the specimen is loaded in compression, tension, or flexure, is approximately 55% of the static strength. Using this conclusion, an S-N curve for plain concrete would be approximately as shown in Fig. 5.18. Since deck cracking due to bending varies inversely with the section modulus, it thus varies inversely with the square of the deck thickness D. Table 5.13 shows how D^2 and a normalized D^2 vary for deck thickness of 6 to 9 inches. Using the values in Table 5.12, if a LL condition on a 7'' deck produced a S_{max}/f_r ratio of 0.8 (f_r=modulus of rupture), then for a 8'' thick deck, the S_{max}/fr ratio would be 0.76x0.8=0.61 (see last column of Table 5.13). As can be seen by the dotted lines in Fig. 5.18, the cycles to failure would be increased enormously (almost 1000 times greater) by such a change. Obviously additional deck cracking and crack propagation due to concrete fatigue will be greatly reduced by an increase in deck thickness. Due to the flatness of S-N curves, at the low stress levels, small decreases in stress can result in enormous increases in fatigue/service life. This is reflected in Fig 5.19 by the steep slope of the fatigue life ratio curve, i.e., the $N/N^{6''}$ curve. Fig 5.19 also shows the deck $(\sigma_{LL/\text{"Fatigue"}})/(\sigma_{LL}^{6''}{}_{/\text{"Fatigue"}})$ variation with deck thickness. In this figure, "fatigue" is shown in quotations because the fatigue life depends on the mean stress level (σ_{DL}) as well as the alternating stress (σ_{LL}).

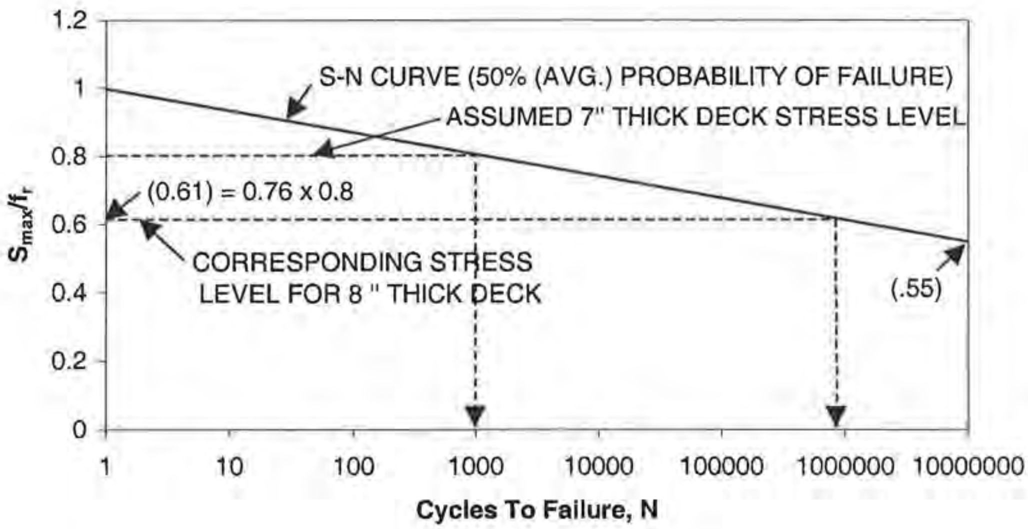


Figure 5.18 Approximate S-N Curve for Concrete

Table 5.13 Deck Fatigue Stresses Normalized w.r.t. 6'' and 7'' Deck Thicknesses

D	D^2 (in^2)	$\dfrac{D^2}{D^2_{6''}}$	$\dfrac{1}{D^2}$ (in^{-2})	$\dfrac{1}{D^2}/\dfrac{1}{D^2_{6''}}$	$\dfrac{1}{D^2}/\dfrac{1}{D^2_{7''}}$
6''	36	1.00	0.0278	1.00	-
7''	49	1.36	0.0204	0.73	1.00
8''	64	1.78	0.0156	0.56	0.76
9''	81	2.25	0.0123	0.44	0.60

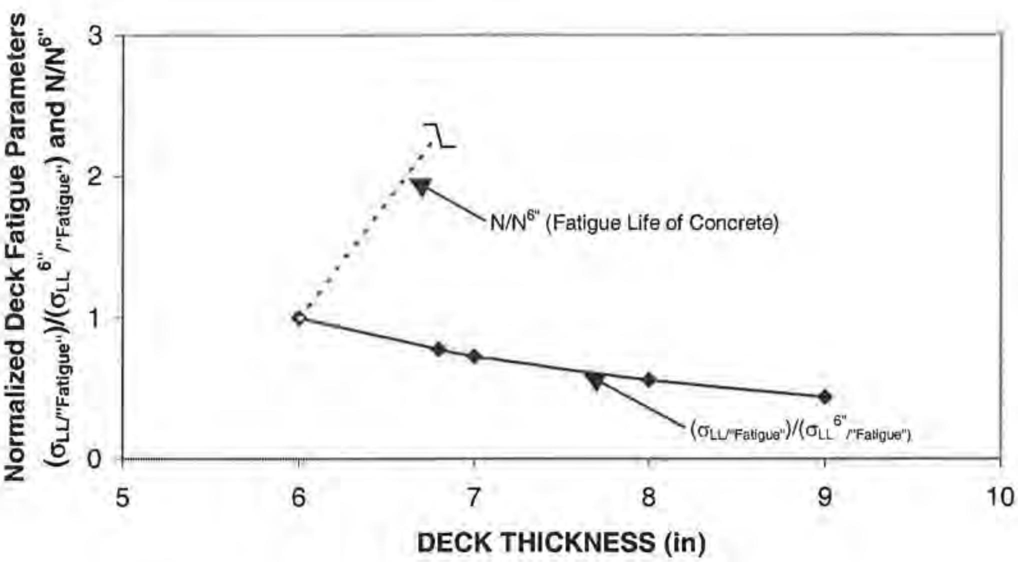


Figure 5.19 Deck Fatigue Life

5.9 Deck/Bridge Costs

To estimate the sensitivity of deck/bridge cost to deck thickness, we must look beyond just theoretical considerations to what bridge design and construction engineers have to say and to logic and common sense. Construction contractors tell us (see Chapter 7) that increasing deck thickness by 1'' (from 7'' to 8'') would increase the deck cost by approximately $\$0.20/ft^2$ or $\$1.80/yd^2$. Bridge design engineers indicate that rarely would a 1'' increase in deck thickness translate into a modified (heavier) design for the bridge support girders, bents, abutments and foundation elements. If this were the case, an increase in deck thickness by 1'' would translate into an increase in deck/bridge initial cost of 2%-3%. A 3% increase in initial cost is shown in Fig.5.20 as a 1.03 value on the normalized scale.

Based on the thicker deck providing reduced stresses and superior performance in all categories examined in this chapter, it is reasonable to assume that an increase in deck thickness from 7'' to 8'' would increase the deck service life a minimum of 10% and quite possibly as much as 50%. If one assumes a current service life for a 7'' deck of 50 years, this would translate into increases in deck service life to between 50 x 1.10 =55 years and 50 x 1.50 = 75 years. Fig. 5.20 shows this assumed range of increase in service life of 10% to 50%, or 1.1 to 1.5 on the normalized (w.r.t. 7'' thick deck) scale (top shaded sector). The reciprocal of these increases in normalized service life values plus the increase in initial cost provide a first order estimate of the normalized life cycle cost of the increase in deck thickness. That is, for 10% (5 years) and 50% (25 years) increases in service life, the associated normalized life cycle cost would be,

$$\text{Normalized Life Cycle Cost} = \frac{1}{1.1} + 0.03$$

$$= 0.91 + 0.03 = 0.94$$

$$\text{Normalized Life Cycle Cost} = \frac{1}{1.5} + 0.03 = 0.70$$

$$= 0.67 + 0.03 = 0.70$$

These values are shown plotted in Fig. 5.20 as a first order estimate of the range of normalized (w.r.t. 7'' thick deck) life cycle cost (the lower darker shaded sector).

If one considers the reduced stresses, deflections, cracking, and improved fatigue life if the deck thickness is increased from 7'' to 8'', it follows that the thicker deck should perform better and require less repairs and maintenance work throughout its life. Also, the high cost of deck repairs, maintenance, and replacement under high volume and concurrent traffic conditions places a premium on placing decks which have a long service life and require less maintenance work. These facts would result in the estimated life cycle cost range shown shaded in Fig. 5.20 shifting even more in favor of using the thicker deck as indicated in Fig. 5.20 by the dotted line bound and lighter shaded lower sector.

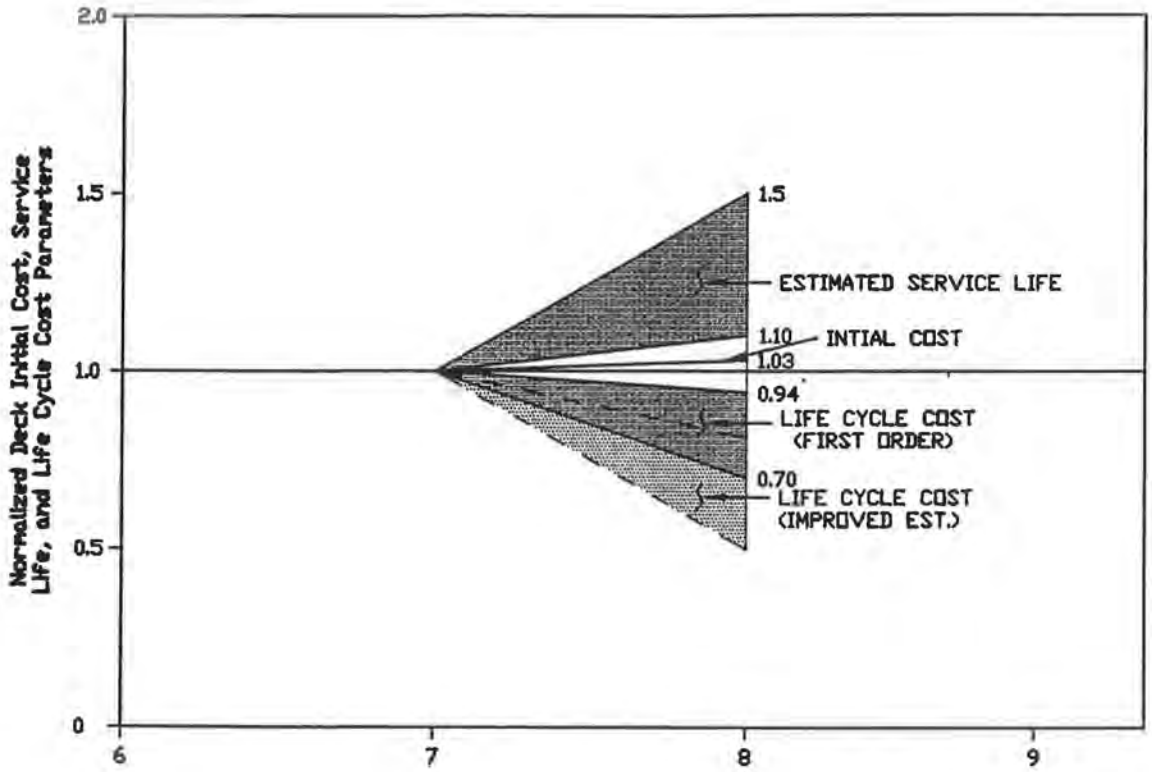


Figure 5.20 Deck/Bridge Estimated Life Cycle Costs

6. EFFECTS OF INCREASING BRIDGE DECK THICKNESS

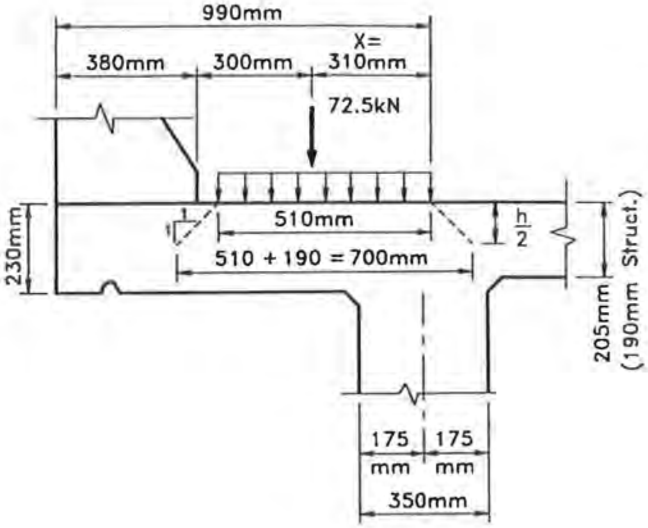
6.1 General

Prior to 1997, the deck thickness of a typical highway bridge in Alabama was around 6½". In 1997 the minimum thickness was increased to 7", and this is the typical thickness now used. However, most states in the U.S. have typical deck thicknesses of around 8". If Alabama were to increase its typical bridge deck thickness from 7" to 8", what would be the primary effects? This is a question of interest and importance to the ALDOT.

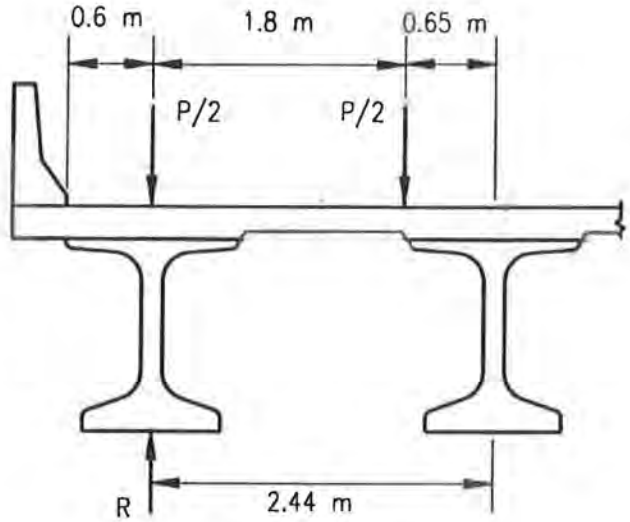
Early discussions with bridge design engineers and analytical analyses indicate that thicker decks should perform better and the additional deck cost should be minimal. However, the additional girder cost to support a thicker and heavier deck may not be minimal, and thus consideration of the effect of deck thickness on the support girder design is of primary interest. The effects of increasing bridge deck thickness in Alabama from 7" to 8" are discussed in the following sections.

6.2 Effect on Support Girders

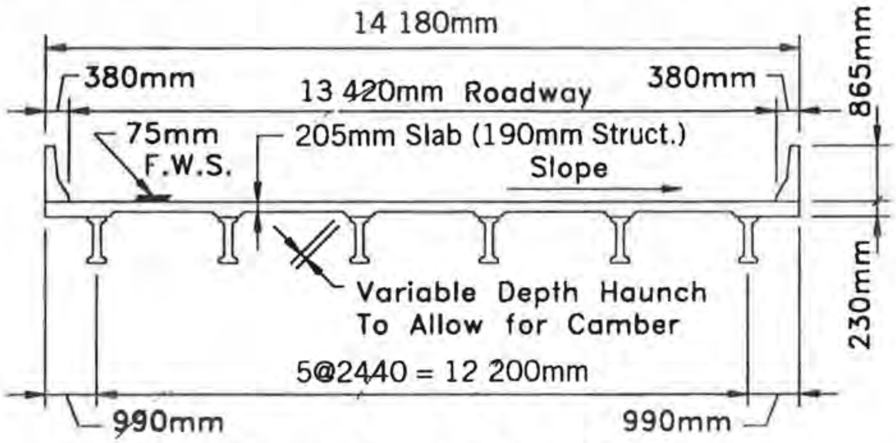
Typical deck-girder bridge cross-sections are shown in Fig. 6.1. As can be seen in that figure the deck thickness "outboard" of the edge girders is thicker than it is between the interior girders because of the cantilever gravity loading of the guard rail and the extreme event loading of a vehicle hitting the guard rail. Thus, the edge girder is already designed for a thickened deck loading acting over approximately one-half its tributary area. Thus, if the entire deck is thickened, the additional deck DL going to the edge girder will be quite small and if the edge girder controls the sizing of the bridge support girders, then the same girders can be used for the entire thickened deck.



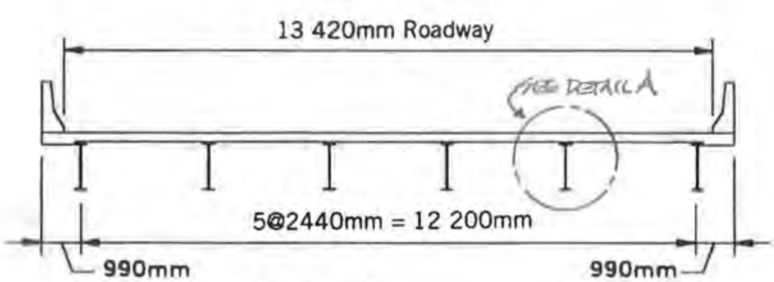
a. Partial T-Girder Sections



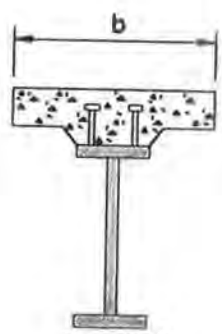
b. Partial Bulb-Tee Sections



c. Prestressed Concrete Girder Sections



d. Steel Girder Section



Detail A

Fig. 6.1 Typical Bridge Cross-Sections

Additionally, most new bridges are now designed using prestressed concrete girders, and many of these use Bulb-Tee sections. Because of their wide flanges, these girders only have a clearance between the edges of the flanges of about one-half the center-to-center spacing of the girders as indicated in Fig. 6.1b. Thus increasing the deck thickness from say 7" to 8" by adding an inch on the underside, i.e., by reducing the haunch thickness, should add negligible additional load to these bridges, and should have negligible effect on the design of the support girders. This is examined more closely in Section 6.6.

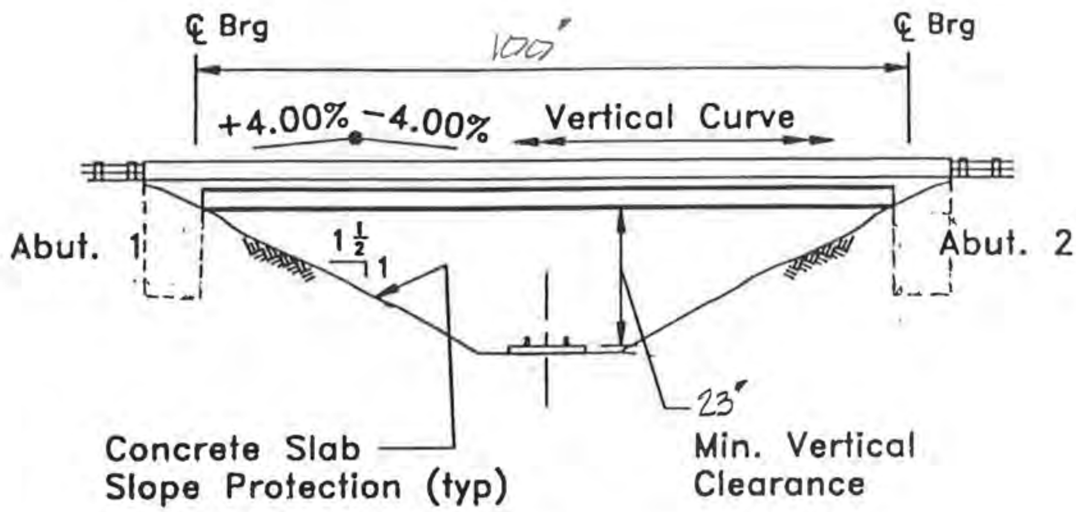
6.3 Increasing Deck Thickness-Example Case

Let us examine the effects of increasing the deck thickness from 7" to 8" on a typical new deck-girder bridge such as the one shown in Fig. 6.2. The bridge is a 100 ft simple span carrying 3 traffic lanes and having a total width of 46'-6". Assume that the original design calls for a 7" deck supported by bulb-tee prestressed girders at 8 ft on center as indicated in Figs. 6.2 and 6.3. Assume that the bridge abutment is supported on a 2 ft thick continuous (transverse to the bridge centerline) spread footing, and that the additional 1" deck thickness can be added to the bottom of the deck as indicated in Fig. 6.3 The primary effects of such an increase in deck thickness are presented and discussed below for the bridge,

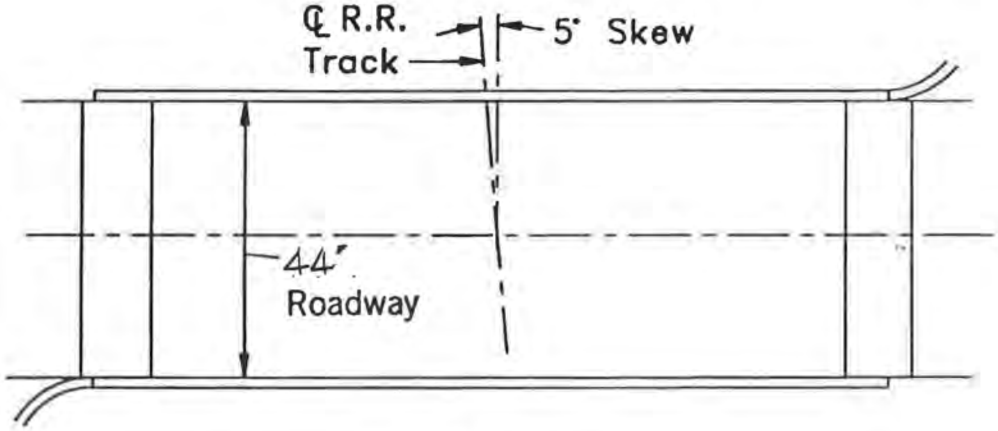
- deck
- support girders
- bearings
- abutments/foundations

Deck. An increase in deck thickness of 1" as indicated in Fig. 6.3 would result in an increase in total deck weight of

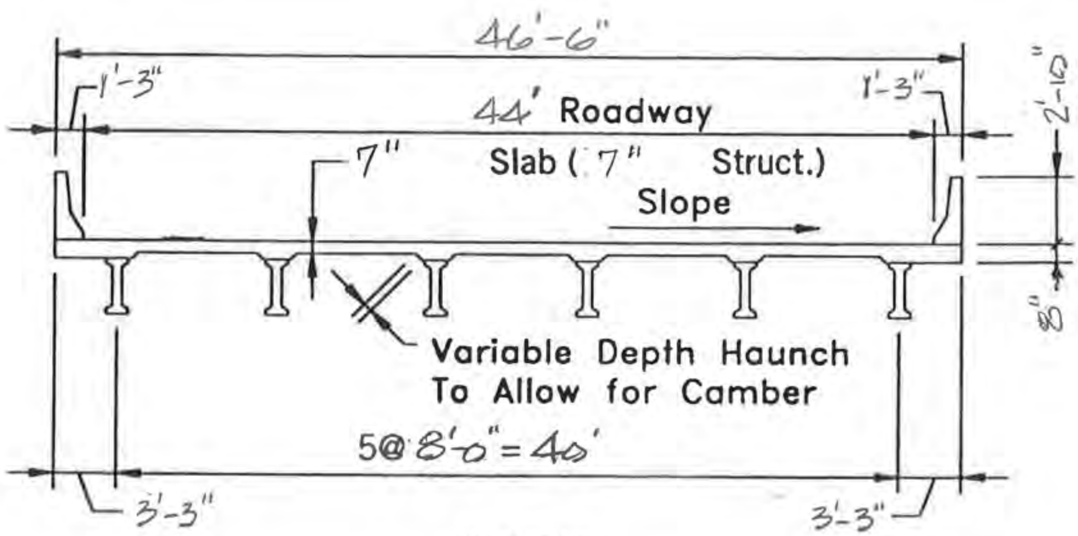
$$\left[\left(4' \ x \ \frac{1'}{12} \ x \ 100'\right) \ (150 \ lb/ft^3)\right] \ (5 \ panels) = 25{,}000 \ lb$$



(a) elevation



(b) plan



(c) section

Fig. 6.2 Prestressed Concrete Girder Bridge Example

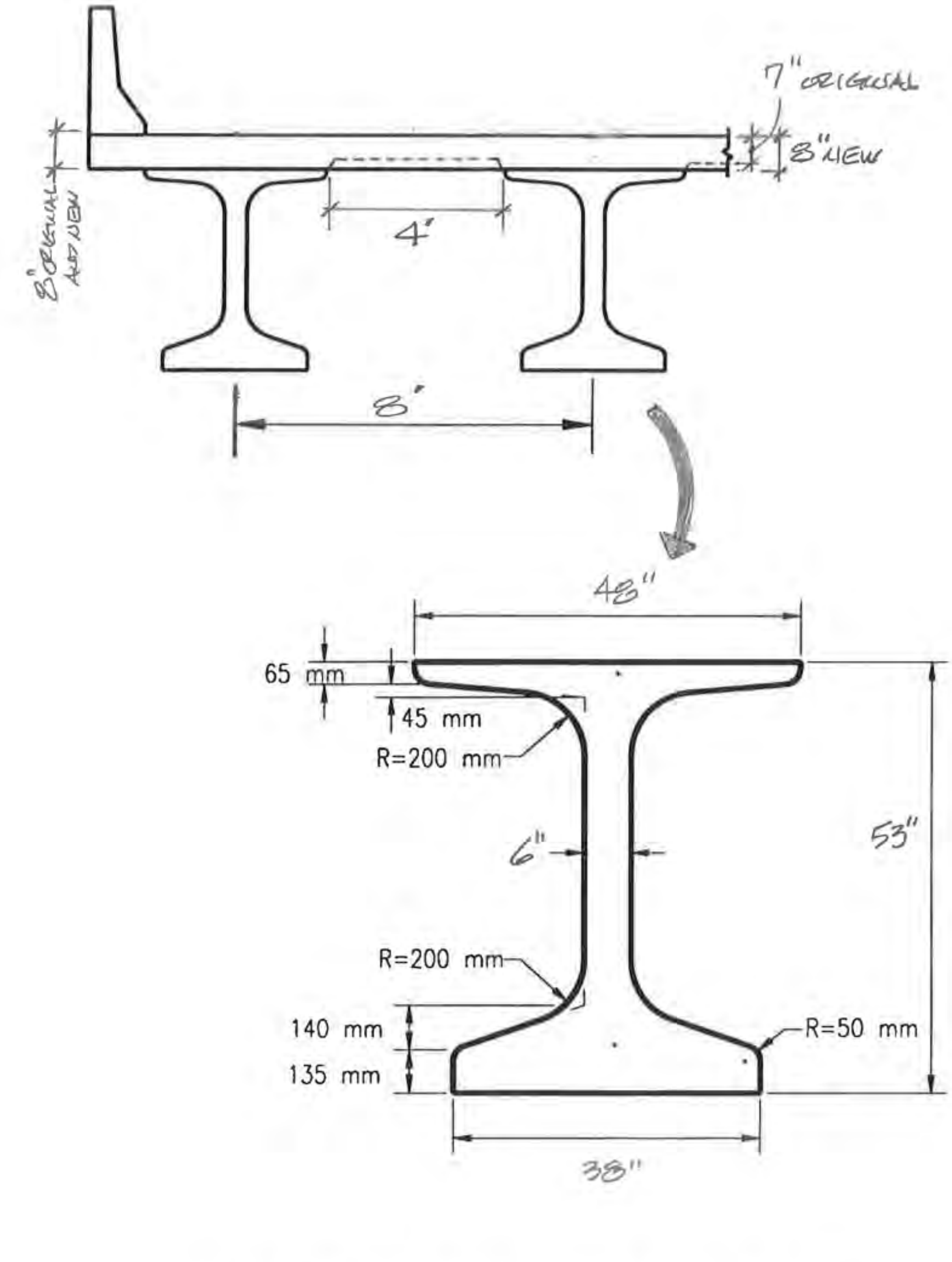


Fig. 6.3 NU Bulb-Tee Girders for Example Bridge

This total additional deck DL would translate to a

- $\Delta w^{DL}_{edge\ girders} = 25\ lb/ft$
- $\Delta R^{DL}_{edge\ girders} = 1{,}250\ lb$
- $\Delta w^{DL}_{interior\ girders} = 50\ lb/ft$
- $\Delta R^{DL}_{interior\ girders} = 2500\ lb$
- $\Delta\ Cost_{Deck} = 4'\ x\ \dfrac{1'}{12}\ x\ 100'\ x\ 5\ x\left(\dfrac{60}{27}\right)\ \$/ft^3 = \$370$

It would also translate into a deck with

- greater stiffness and less deflection
- greater cracking moment resistance
- greater ultimate moment capacity
- greater punching shear resistance
- greater fatigue life

Girders. Since the bridge edge girders normally control the sizing of the support girders, and since increasing the deck thickness from 7" to 8" will only increase the load on the edge girders by 25 lb/ft, the edge girder sizing should not be affected by the additional 1" of deck thickness. The interior girders will have their design DL increased by 50 lb/ft which is still rather small, and additionally they typically do not control the girder sizing. Thus the bridge support girder design/sizing should not be affected by the additional 1" of deck thickness.

Bearings. Interior girder bearings loads will be increased by 2500 lb and exterior girder bearing loads by 1250 lb. Both of these increases should be insignificant in comparison with the total design girder bearing loads. In turn the same bearings could be used with the thicker deck, and thus the additional bearing cost would be zero.

Abutments/Foundations. The additional dead load going to each abutment due to the 1" increase in deck thickness would be as shown in Fig. 6.4.

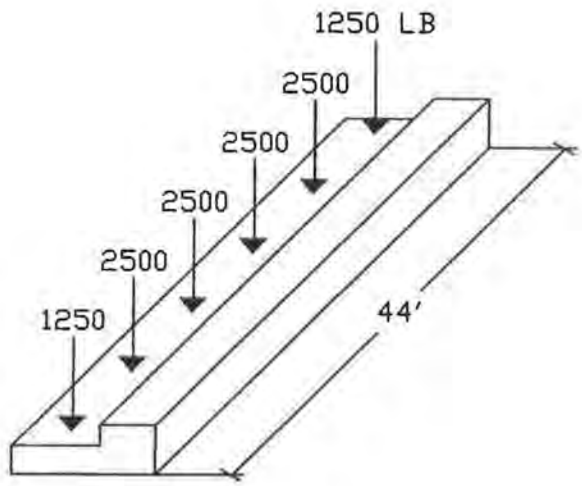


Fig. 6.4. Abutment/Foundation Δ-Dead Load

Assuming a 2 ft thick spread footing under the abutment, and an allowable soil pressure of 2000 psf, the additional area required for the spread footing to support the additional $\Delta R_{abutment}$ would be

$$\Delta A_{Ftg} = \frac{\Delta R_{Abutment}}{2000\ psf} = \frac{12{,}500\ lb}{2000\ lb/ft^2} = 6.25\ ft^2$$

$$\Delta width = \frac{6.25\ ft^2}{44'} = 0.142' = 1.70'' \Rightarrow Use\ 2''$$

$$\Delta COST_{ABUT.\ FTG.} = \left(\frac{2}{12}\right)' x\ 2'\ x\ 44'\ x \left(\frac{60}{27}\right)\ \$/ft^3 = \$32.50$$

$$\Delta COST_{ABUT.\ FTG.\ TOTAL} = 32.50\ x\ 2 = \$65$$

In summary, the additional costs of increasing the deck thickness of the bulb-tee girder bridge shown in Figs. 6.2 and 6.3 from 7" to 8" are shown in Table 6.1. Obviously this small additional cost is negligible, and if it will result in any improvement in bridge performance, particularily in service life, then it would be a change well worth making. Since increasing the deck thickness should significantly improve the deck performance, as indicated earlier, this change should be made.

Table 6.1 Additional Cost of 1"Increase in Deck Thickness of Example Bridge

Bridge Component	Δ Cost $
Guard Rail	0
Deck	\$370
Girders	0
Bearings	0
Abutments/Foundations	\$65
Drains, Scuppers, Joints, Lighting, Paint Striping, Etc.	0
TOTAL	\$435*

$$* \Delta Unit\ Cost = \frac{\$435}{46.5' \times 100'} = \$0.09/s.f.$$

6.4 Effects on Other Bridge Components

Discussions with ALDOT bridge engineers regarding the effects of increasing the thickness of ALDOT's bridge decks from a typical value of 7" to 8" provided the following information and insights.

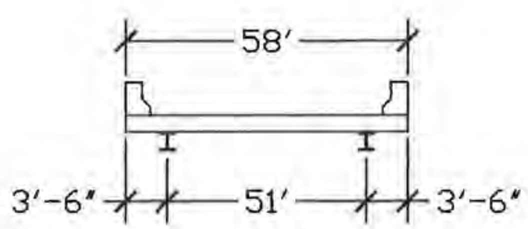
- When using AASHTO, bulb-tee, and steel girders (which covers almost all of their bridges), the thickness of the deck immediately above the girders is typically 9" - 10" for a 7" thick deck (at mid point between girders).
- The additional thickness above the girders, i.e., the deck haunches, is needed to allow the tops of the girder shear studs (typically 4" long) to fall below the deck top rebar mat (to avoid interference), and to provide proper top bar cover. It also provides extra

vertical clearance for variances in girder camber, which is changing with time for prestressed girders due to shrinkage and creep, and fabrication and construction imperfections. The depth of the haunches varies along the length of the girder and this is achieved by adjusting the height of the light weight metal angles attached to the edges of the girder top flange which are used to support the deck SIP metal forms.

- In order to reduce the additional deck dead load resulting from increasing the deck thickness from 7" to 8", the additional 1" could be added to the bottom of the deck as indicated in Fig. 6.5. In the case of bridges with bulb-tee girders, this would result in increasing the deck DL by only about one-half of that of the 1" being added to the top of the deck. As indicated above and in Fig. 6.6, the critical deck thickness is that immediately above the center line of the girders.

- Minimum, maximum, and norm values for ALDOT deck extensions beyond the exterior girder are as indicated in Fig. 6.6. Because the overhang norm is 3' - 4½" and because they desire to use the same size girder for interior and exterior girders, ALDOT seeks to use a girder spacing of approximately 7 ft where feasible. This results in the effective composite beam width being approximately the same for both interior and exterior girders. This yields approximately the same design for these girders and thus allows the same girder to be efficiently used for both interior and exterior girders.

- If, based on number of traffic lanes required, shoulder widths, and Jersey side barriers, a bridge horizontal section dimensions were as shown below, then potential candidate girder spacings would be as indicated below. ALDOT bridge engineers would select the 8 girders at 7'-3½" spacing as the preliminary design for the reasons cited above. Depending on the bridge span geometry, underneath clearances and vertical elevations, they may need to adjust the number/spacing of the girders up or down once they go into the detailed design phase.



Girder Spacings:

$$\frac{51'}{6} = 8'\text{-}6'' \quad (7\ \textit{girders})$$

$$\frac{51'}{7} = 7'\text{-}3\tfrac{1}{2}'' \quad (8\ \textit{girders})$$

$$\frac{51'}{8} = 6'\text{-}4\tfrac{1}{2}'' \quad (9\ \textit{girders})$$

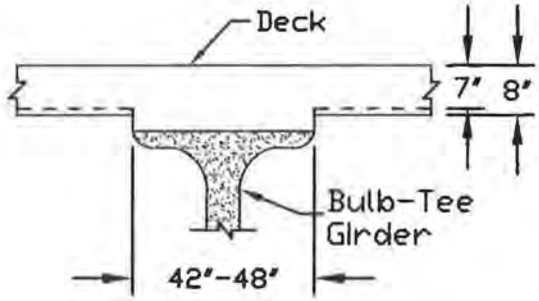


Fig. 6.5 Deck and Bulb-Tee Girder Section

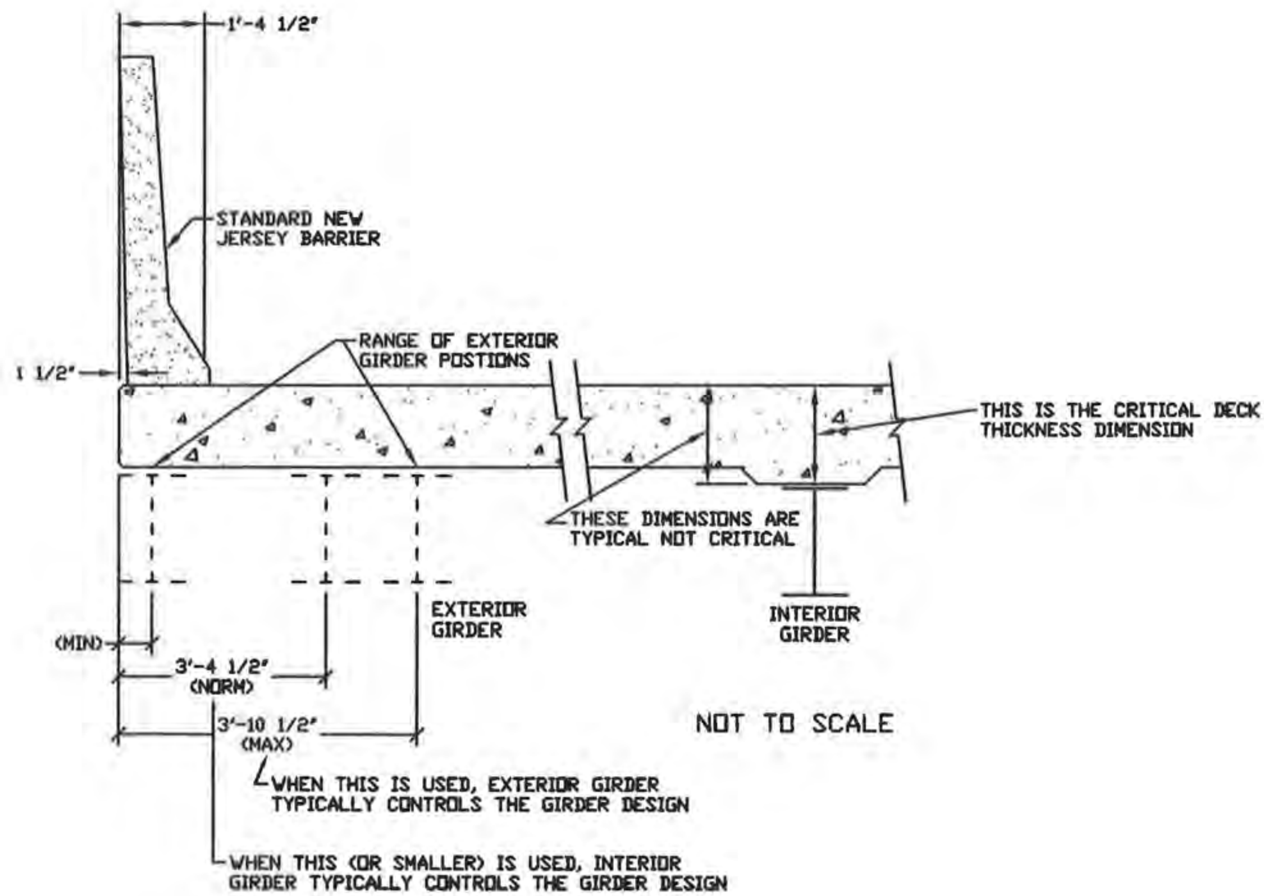


Fig. 6.6 ALDOT Typical Deck-Girder Layout and Geometrics

- In deciding the bridge girder spacing and deck thickness, the ALDOT makes no attempt to achieve a particular deck S/D ratio, but rather they select a deck thickness that satisfies the AASHTO requirements of

$$D_{\min} \geq 175 \ mm \quad (\approx 7'')$$

and

$$D_{\min} \geq \frac{S + 3000}{30} \quad \text{(to control deck deflections between girders)}$$

Note that if an 8" thick deck is employed, the first expression will be satisfied, and the second expression yields,

$$S_{max} = 30\ (8'' \times 25.4\ \text{mm/in}) - 3000$$

$$= 6096 - 3000 = 3096\ \text{mm}$$

$$= 10'\text{-}2''$$

At the present time, the ALDOT never uses girder spacings in excess of 10 feet, and thus an 8" deck thickness would suffice for all bridge girder spacings currently employed by the ALDOT.

- The ALDOT uses the same girder all across the width of their bridges. Some books/articles in the literature indicate that the edge girders normally control the sizing of the girders (but only slightly) due to the extreme loading condition of a truck hitting the bridge guard rail. However, ALDOT bridge engineers indicate that the edge girders have a reduced LL acting on them and a reduced distribution factor (see Table 6.2), and that the interior girders typically control the girder sizing for typical deck overhangs (beyond the exterior girder). For maximum deck overhang, the exterior girders typically control the girder sizing as indicated in Fig. 6.6.

- If the deck of a typical concrete deck-girder bridge were increased from 7" to 8", the primary effects of this change on the major bridge components would be as indicated in Table 6.3.

- As noted in bold print in Table 6.3, by far the two largest and most significant impacts of increasing the bridge deck thickness from 7" to 8" are,

  (1) significantly enhance strength, stiffness, and durability of the deck itself at almost no additional cost of the deck.

Table 6.2 AASHTO Wheel Load Distributions in Longitudinal Girders for Bridges with Concrete Decks (modified from Ref. 31)

TYPE OF GIRDERS	BRIDGE TRAFFIC LANES		NOTES
	ONE	TWO OR MORE	
INTERIOR GIRDERS[1]			
Steel I-Beam Stringers and Prestressed Concrete Girders	$\frac{S}{7.0}$ If S exceeds 10 ft, see note this row.	$\frac{S}{5.5}$ If S exceeds 14 ft, see note this row.	Assume the flooring between stringer acts as a simple beam with the load on each stringer being the wheel load reaction.
Concrete T-Beams	$\frac{S}{6.5}$ If S exceeds 6 ft, see note this row.	$\frac{S}{6.0}$ If S exceeds 10 ft, see note this row.	Assume the flooring between stringers acts as a simple beam with the load on each stringer being the wheel load reaction.
Timber Stringers	$\frac{S}{6.0}$ If S exceeds 6 ft, see note this row.	$\frac{S}{5.0}$ If S exceeds 10 ft, see note this row.	Assume the flooring between stringers acts as a simple beam with the load on each stringer being the wheel load reaction.
Concrete Box Girders	$\frac{S}{8.0}$ If S exceeds 12 ft, see note this row.	$\frac{S}{7.0}$ If S exceeds 16 ft, see note this row.	Assume the flooring between stringers acts as a simple beam...as above. Omit sidewalk live load for interior and exterior girders designed with this criteria.
Steel Box Girders	Find live load bending moment for each girder using: FRACTION OF WHEEL LOAD = 0.1 + 1.7R + 0.85/NW		R = NW/Number of Box Girders ($0.5 < R < 1.5$) NW = WC/12 reduced to the nearest whole number WC = Curb to curb or barrier to barrier width (feet)
Prestressed Concrete Spread Box Beams	Find interior girder live load bending moment using: FRACTION OF WHEEL LOAD = (2NL/NB) + k(S/L) For exterior girder assume flooring between stringers to act as a simple beam...as above, but not less than 2NL/NB.		NL = Number of Design Traffic Lanes NB = Number of Beams ($4 \leq NB \leq 10$) S = Beam Spacing ($6.57 \leq S \leq 11.00$) (feet) L = Span Length (feet); W = Curb to Curb Width (feet) k = 0.07W - NL(0.10NL - 0.26) - 0.20NB - 0.12
EXTERIOR GIRDERS[2]			
Bridges with 4 or More Girders	$DF = \frac{S}{5.5} \quad (S \leq 6')$ or $DF = \frac{S}{4.0 + 0.25S} \quad (6' \leq S \leq 14')$		For S ≤ 6', the DF is the same as for Interior Girders. For S> 6', the DF is less than that for Interior Girders.

[1] S = Average Stringer Spacing in feet.
[2] S = Distance between Exterior and Adjacent Interior Girder in feet.

Table 6.3 Primary Effects of Increasing the Deck Thickness from 7" to 8" on Bridge Components

Bridge Component	Change in Design Conditions/Requirements	Probable Change in Component Sizing
Deck	• Increase in deck DL = 12 psf • If add to bottom of deck for bulb-tee girders, the 12 psf increase would just be applied to about one-half of the deck width or surface area. • SIP metal forms support the deck DL and need to be checked for adequacy. • Allows the use of 2½" top cover if so desired.	• Small increase in transverse rebar spacing (due to increased effective depth, d) • Use same SIP metal forms as now using • Increase top bar cover to 2½" (recommended).
Girders	• Additional deck DL from about 48 lb/ft for bulb-tee girders to 84 lb/ft for steel WF girders for an 8' girder spacing. • Larger deck-girder composite section which would provide additional strength and stiffness for LLs.	• The small additional deck DL may require using a slightly shorter girder span, an extra line of girders, or an increased girder size if the girder size for a 7" deck is already at it's limiting length. This will rarely be the case. • In most cases it will not effect girder sizing at all, but in rare cases it could require changes as indicated above. • The effect on the girder sizing and design is by far the largest and most significant effect that increasing the deck thickness will have (other than enhancing the deck itself).
Bearings	Slightly heavier design dead load per bearing	None
Abutments/ Bents	Slightly heavier design dead load	None
Foundations	Slightly heavier design dead load	None - to slightly wider footing (≈ 2" wider)

(2) potential for an increase in the girder cost via the possible need to add a line of girders, use somewhat larger girders, or reduce the girder span length. Obviously this potential negative impact needs to be looked into more closely, and this is examined by a Girder Cost Example in the following section.

- Approximate maximum span and unit costs of prestressed concrete girders currently (November 1999) used by the ALDOT for deck-girder bridges are shown in Table 6.4. It should be noted that ALDOT does not currently use AASHTO Type IV girders, but employs the BT-54 instead. Also, as indicated in the first footnote in Table 6.4, the current total unit cost for prestressed girder-deck bridges in Alabama is approximately \$35 - \$40/sf. This appears to be one of the lowest bridge unit costs in the country.

Table 6.4 Approximate Maximum Span and Unit Cost of Prestressed Concrete Girders for Deck-Girder Bridges in Alabama.[1]

Girder Type	Approx. Max Span[2]	Approx. Unit Cost
AASHTO Type I	45'	\$55/ft.
AASHTO Type II	62'	\$65/ft.
AASHTO Type III	85'	\$80/ft.
BT - 54	105'	\$95/ft.
BT - 63	120'	\$110/ft.
BT - 72	140'	\$135/ft.

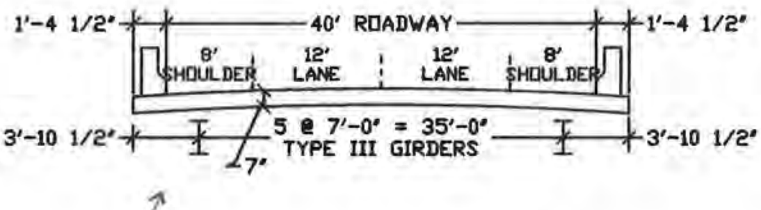
[1]Approximate total bridge unit cost for prestressed girder-deck bridges in Alabama is \$35-\$40/ft.[2].
[2]These approximate maximum spans are based on relatively small girder spacings.

6.5 Girder Cost Example

A girder cost example is provided below to gain a better perspective on the possible impact of increasing the bridge deck thickness by 1" on girder and overall bridge costs.

EXAMPLE:

Problem Statement: Assume the deck-girder section shown below is the final design for a 75' simple span bridge of composite design in Alabama. Determine the costs associated with this design, and the maximum probable increase in cost if the deck thickness is increased to 8".



With Max Overhang Being Used, The Exterior Girders Probably Control The Girder Sizing

Partial Cost Analysis:

Mid-Range Value

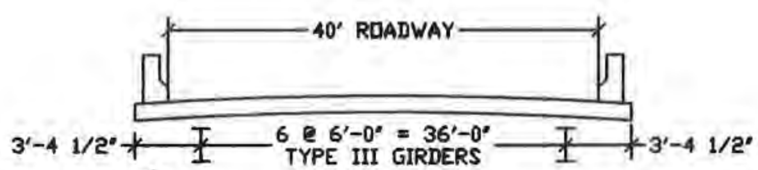
Total Bridge Cost = 42.75' x 75' x \$37.50/ft^2
= \$120,230

Girder Cost = 6 x 75' x \$80/ft
= \$36,000

Thus the girder cost is 29.9% of the total cost of the bridge.

Increase Deck Thickness:

Assume now that the deck thickness above is increased by 1" to 8". Further assume a worst case scenerio where the resulting increase in DL requires using a 7th girder as shown below.



With This Overhang, The Interior Girders Probably Control The Girder Sizing.

For this case,

$$\Delta \text{ Cost} = 75' \times \$80/\text{ft} = \$6000$$

or a 5% increase in initial bridge cost. However, the extra girder would result in girders being at 6' on center, and should result in a stiffer and stronger superstructure with a greater fatigue life relative to the 6 girder and 7'-0" spacing design. This in turn should result in lower maintenance cost, greater service life, and lower life-cycle cost.

In the example above, it was assumed that the small additional DL moment resulting from the 1" increase in deck thickness would be sufficient to require an extra girder. As seen above, even in this case, the effect of the deck thickness increase was not really significant if one considers the life-cycle cost of the bridge. In discussions with ALDOT bridge design engineers, it is rare when a bridge girder design is on the "bubble" of its limiting capacity where a 1" addition in deck thickness would require going to a shorter span, closer girder spacing, or a larger girder. Thus, for the typical case, an increase in deck thickness of 1" would not affect the bridge girder design.

6.6 Results of Girder Design Examples

Results of the deck thickness survey, as well as discussions with ALDOT's bridge design engineers, indicated that the potential requirement for increased support girder load capacity and cost is the primary concern of increasing deck thickness. To assess the effect of increasing the deck thickness by 1" on the support girder design for deck - girder bridges, five example girder designs were examined. All 5 designs were for simple span bridges with one each being

- Rolled steel girder, noncomposite, short span - Example B1
- Rolled steel girder, composite, short span - Example B2
- Rolled steel girder, composite, intermediate span - Example - B3

- AASHTO Type III PCI girder, composite, intermediate span - Example B4
- Rolled steel girder, noncomposite, long span - Example B5

All except Example 5 were taken from the literature (6,31), and each example is given in Appendix B.

In each case, the design examples were modified via increasing the deck thickness by 1" and re-evaluating the design parameter values and girder sizing. The resulting girder maximum design moments were increased by 1.57 - 3.14% depending on the particular example/bridge, and the resulting girder sizing was the same in all 5 examples. That is, the 1" increase in bridge deck thickness did not require heavier girders in any of the examples. The primary results of the five design examples are summarized in Table 6.5.

The design examples examined are believed to be typical, and thus increasing the deck thickness by 1" should not typically affect the girder sizing. There will of course be instances where a 1" thicker deck will require a somewhat larger girder section, closer girder spacing, or shorter girder span length as indicated in the previous subsection. However, these instances will be the exception rather than the rule. For these exception cases, whereas the initial costs of the bridges will be increased, it is not clear that the life-cycle-costs of the bridges will be increased, and the authors believe that the life-cycle-cost of these exceptional cases will also be decreased.

Table 6.5 Summary of Effects of Increasing Deck Thickness by 1" on Girder Design Moments, Shears, and Member Sizings for Five Design Examples

Example/Bridge Description	M_{max}	V_{max}	Girder Selected	Comments
Example 1. Noncomposite, SS, Steel Girder Bridge. LRFD Specs. Deck t = 205 mm ($8^1/_{16}$") Span Length = 10.5m (34.5') Girder Spacing = 2.44 m (8.0') Example 1a. Same bridge as Ex. 1, but with Deck t = 230 mm ($9^1/_{16}$")	1405 kNm (factored) 1427 kNm (factored)	613 kN (factored) 623 kN (factored)	W760 x 134 (W30 x 90) W760 x 134 (W30 x 90)	See App. B Girder M_{max} increased by 1.57%, V_{max} and R_{max} increased by 1.63%, and no change in girder size for a 1" increase in deck thickness.
Example 2. Composite, SS, Steel Girder Bridge. LRFD Specs. Deck t = 205 mm ($8^1/_{16}$") Span Length = 10.5 m (34.5') Girder Spacing = 2.44 m (8.0') Example 2a. Same bridge as Ex. 2, but with Deck t = 230 mm ($9^1/_{16}$")	1334 kNm (factored) 1359 kNm (factored)	625 kN (factored) 633 kN (factored)	W610 x 101 with 10 mm x 200 mm C.P. W610 x 101with 10 mm x 200 mm C.P.	See App. B Girder M_{max} increased by 1.87%, V_{max} and R_{max} increased by 1.28%, and no change in girder size for a 1" increase in deck thickness
Example 3. Composite, SS, Steel Girder Bridge. WSD Specs. Deck t = 8" Span Length = 45' Girder Spacing = 8' Example 3a. Same bridge as Ex. 3, but with Deck t = 9"	$828^{\prime k}$ (unfactored) $854^{\prime k}$ (unfactored)	– –	W33 x 130 W33 x 130	See App. B Girder M_{max} increased by 3.14%, and no change in girder size for a 1" increase in deck thickness
Example 4. Composite, SS, Type III AASHTO-PCI Girder Bridge. WSD Specs. Deck t = 7" Span Length = 70' Girder Spacing = 7' - 3" Example 4a. Same bridge as Ex. 4, but with Deck t = 8"	$1795^{\prime k}$ (unfactored) $1850^{\prime k}$ (unfactored)	– –	Type III Girder Type III Girder	Girder M_{max} increased by 3.06%, and no change in girder size for a 1" increase in deck thickness
Example 5. Noncomposite, SS, Steel Girder Bridge. LRFD Specs. Deck t= 205 mm ($8^1/_{16}$") Span Length = 30.5 m (100') Girder Spacing = 2.44 m (8.0') Example 5a. Same bridge as Ex.5, but with Deck t = 230 mm ($9^1/_{16}$")	8332 kNm (factored) 8522 kNm (factored)	1131 kN (factored) 1157 kN (factored)	W1100 x 499 (W44 x 335) W1100 x 499 (W44 x 335)	See App. B Girder M_{max} increased by 2.28%, and V_{max} and R_{max} increased by 2.30%, and no change in girder size for a 1" increase in deck thickness

7. CONSTRUCTION AND COST CONSIDERATIONS - CONTRACTOR PERSPECTIVE

7.1 General

In late August and early September 1999, the project PI met with 3 Alabama bridge contractors to discuss and identify the primary effects of increasing deck thickness on the construction and cost of decks and other bridge components. Also, the relative merits/demerits of ALDOT employing a standard deck thickness of 8" were discussed from construction and cost view points. A standard deck thickness, e.g. 8", should allow standardization of rebar mat support chairs in attaining proper bottom and top covers. It should simplify construction and owner inspection, and should result in enhanced deck quality/durability.

Prior to meeting with the bridge contractors, discussions via phone with some contractors and with ALDOT bridge construction personnel, typical bridge deck construction tasks and cost items were identified and these are listed below.

1. SIP metal forms (material)
2. Placing SIP metal forms between girders
3. Forming deck overhang beyond edge girders
*4. Deck rebar (material)
*5. Placing deck rebar
6. Guard rail rebar (material)
7. Placing guard rail rebar
8. Expansion/contraction joint assemblies (material)
9. Placing expansion contraction joints
10. Setting concrete screed rails and screed unit in place and making dry-runs
*11. Deck concrete (material)
*12. Placing, compacting, and screeding deck concrete
13. Final finishing of deck concrete
14. Apply and maintain curing system (7 days) on deck concrete
15. Finish placing guard rail rebar
16. Form guard rail

17. Guard rail concrete (material)
18. Place guard rail concrete
19. Strip guard rail forms
20. Strip deck forms under edge overhangs
21. Rub/touch-up guard rail and deck overhangs to remove blemishes
22. Groove deck top surface

It should be noted that only the items marked with an asterick (*) in the above listing would be impacted by changing the deck thickness. If ALDOT were to adopt an 8" "standard" deck thickness, the rebar quantities (Items 4 and 5) would decrease relative to currently used values, and the concrete quantities (Items 11 and 12) would increase relative to currently used values. The net effect on the cost of the deck would probably be about nil (see example in Appendix C). However, the 8" deck would be heavier (approximately 0-12 psf heavier depending on location on the deck) than those currently used and thus could require somewhat heavier girders, bents, abutments and foundations. It is estimated that this nominal increase in deck weight would not significantly alter the sizing and weight of the girders, bents and foundation elements. This is discussed and verified by design examples in Chapter 6 and Appendix B.

7.2 Contractor Meeting Results

The bridge contractors and persons met with were:

Mr. Keith Mims, President
Alabama Bridge Builders, Inc.
Highway 79 North
P.O. Box 1000
Pinson, AL 35126

Mr. Tim McInnis, Vice President
McInnis Corporation
1120 Parker Street
P.O. Box 9423
Montgomery, AL 36108-0423

Mr. Charles Davis, Jr., Chief Engineer
Scott Bridge Company, Inc.
614 Second Avenue
P.O. Box 2000
Opelika, AL 36803

Results of the meetings are summarized below.

7.2.1 Alabama Bridge Builders, Inc. The PI met with Mr. Keith (Tack) Mims, at his office in Pinson, AL on August 23, 1999 to discuss the effects of increasing the thickness of bridge decks from construction and cost points of view. During the course of our discussions, Mr. Mims expressed his opinions and recommendations on bridge construction issues other than those pertaining to the deck, and these were encouraged by the PI. Listed below are the primary observations and comments by Mr. Mims.

- Going to thicker decks is needed and is a good idea
- Going to 2½" top cover will help assure getting a minimum of 2". He feels that ALDOT and others overestimate the deck DL deflection and this results in a top cover less than the current specified value of 2". The 2½" cover would also allow for the possibility of future grinding of the deck for rideability.
- Contractors are having difficulty getting 4 ksi concrete with ALDOT's current mixture and specifications.
- They have difficulty staying in the 4% - 6% specs on air content. They may be in specs when a truck leaves batch plant, but out of spec when it gets to job site, or in specs at beginning of unloading a truck, and out of specs for the last yd^3 in the truck. They indicate that they particularly have trouble when additional AE is added at the job site. When this is done, air content is then usually too high and the concrete is understrength. They indicate difference in air content testing results between testing fresh concrete as opposed to hardened concrete. They also question the repeatability of just the basic fresh concrete air content test.
- Alabama Bridge Builders would strongly prefer that the ALDOT dictate the concrete mixture to use. They are not experts on mixture design, and when given a lot of latitude in this area they view that

they end up being the middle-man between the concrete producers/suppliers and the ALDOT engineers. They are uncomfortable in this middle-man role. However, they feel that the mixture design dictated by ALDOT should be for 5000 psi concrete if they need/require 4000 psi concrete for their decks. They make the point that when decks have to be torn out and replaced due to the concrete being understrength, then future bids will include an adjustment for this and the cost will be passed along to ALDOT and the tax payers. It would be cheaper to use a little more expensive concrete and avoid having to tear out new decks.

- They view that almost any and all standardization would help construction quality and reduce cost. They recommend standardization of

  - concrete mixture design
  - prestress girder stranding
  - deck thickness
  - deck rebar
  - girder spacing

- The only additional deck cost estimated with going to thicker bridge decks would be the cost of the additional concrete. He indicated that the additional cost to place the additional concrete would be negligible, and could range from 0 - \$10/yd^3. He also pointed out that most new bridges are utilizing bulb-T girders with 9" - 10" deck thickness above the girders, and due to their large flange width, increasing the deck thickness from say 7" to 8" would only result in approximately a 1/2" average thickness increase over the entire deck. Thus, the additional weight/load and cost from such an increase should be almost negligible.

- It would be very helpful to the contractor to use a high slump concrete (via use of superplasticizers) in placing bridge decks. He viewed that the use of superplasticizers was absolutely essential for placing web walls (diaphragms) and abutment end walls. He viewed that you couldn't get 3½" - 4" slump concrete in these forms.

- He indicated that the primary factor affecting their decision to place deck concrete by crane-and-bucket or by pumping was if a crane was already available at the site. If a crane was available then they would typically chose to place by crane-and bucket, using a 2 yd^3 bucket. He indicated that they could unload a concrete truck (6 - 10 yd^3) in 15 - 18 minutes via crane-and-bucket placement, and that this was their targeted time allowance. The negative of placement by pumping is the cost of hiring a pumper truck.

- They see a lot of merit in ALDOT requiring bridge deck placements during the summer months to be conducted at say 7:00 p.m. - 11:00 p.m. at night (temperature goes down, humidity goes up, and wind goes down - all of these are good when placing concrete). This would allow them more time to properly place, consolidate, and finish the concrete, and to set-up their curing system. They would prefer such a placement requirement and believe that it would result in a better finished product. However, they recognize that it will cost a little more (night lighting, maybe higher unit cost of concrete, etc), and thus they view that ALDOT should look at this and make a decision regarding night placement of bridge decks in the summer months, and require it of all contractors if they so decide.

- Alabama Bridge Builders, helped place a Poly-Carb epoxy based overlay on 2 bridges on I-20 near Pell City, AL about 6 - 7 years ago to improve the skid resistance of those bridges. These overlays have and are performing great and are not showing any signs of distress or deterioration. The bridges are 3-span continuous reinforced concrete haunched T-beam design that was widely used in the late 1960's. Mr. Mims believes that such overlays are the answer to the current interstate bridge deck problems in Birmingham. He would like to place some of these overlays on the Birmingham bridges on a test basis, and would like to develop special bridge deck overlaying capabilities and expertise. Having experienced bridge contractors with such specialty expertise would be good for ALDOT and Alabama.

7.2.2 Scott Bridge Company, Inc. The PI met with Mr. Chuck Davis, Chief Engineer of Scott Bridge Company, at his office in Opelika, AL on August 31, 1999 to discuss the effects of increasing the thickness of bridge decks from construction and cost points of view. Other topics pertaining to bridge deck construction and cost and enhancing durability and service life were also discussed. Listed below are the primary observations and comments by Mr. Davis.

- Going to thicker decks in a good idea and should provide an improvement in deck service life.

- Additional deck cost would be the cost of the additional concrete at

$60/yd^3. For a 1" increase in deck thickness this would translate into an additional cost of approximately $1.70 per square yard of deck.

- An additional deck thickness of 1" would weigh an additional 12 psf and this may or may not require heavier and more expensive support girders and substructures.

- Standardization in bridge geometry, details, materials, etc. would in general be very helpful in improving construction quality and in reducing construction time and cost.

- Standardization in girder spacing, and in turn bridge diaphragms, would probably be the most helpful to the contractor.

- Scott Bridge has not experienced any problems getting 4000 psi concrete with ALDOT's mixture design, nor staying in the 4% - 6% specs on air content.

- Scott Bridge strongly prefers allowing the contractor the freedom to develop his own concrete mixture. Mr. Davis indicated that some ready-mix plants do not have good mixture designs to draw from, but that most do. He recommended that ALDOT include a mixture design in the specs, but that they allow an alternative mixture to be used (with their approval). He indicated ALDOT may want to require the contractor to provide a trial batch (maybe a ½ truck load) of any alternate mixture delivered to the job site for testing as part of the approval process.

- Use of superplasticizers in deck concrete would be very helpful in improving construction quality and in reducing construction time and cost. He indicated that superplasticizers are supposed to have a zero slump loss during the first hour, and a reasonable rate of slump loss after that. He indicated that the MSDOT required the use of superplasticizer in Scott's most recent job for them. They added the superplasticizer at the job site and it worked beautifully.

- Scott Bridge is comfortable placing bridge decks by pumping or by crane-and-bucket. The biggest negatives for pumping are additional cost for pump truck, and possible pump truck/line breakdown. When they place by crane-and-bucket, they typically use a 5 yd^3 bucket and fill it with about 4 yd^3 per placement thus emptying an 8 yd^3 concrete truck in two loads. They feel that placing a deck fairly quickly is important to the quality of the deck. Mr. David indicated that the GADOT requires minimum rates of concrete placement on their bridges. These rates are shown in Table 7.1, and translate into placing all of the concrete in a maximum period of time of approximately 1½-3 hours depending on the size of the deck placement.

TABLE 7.1 GADOT CONCRETE PLACEMENT RATE REQUIREMENTS

500.06 PRODUCTION AND PLACEMENT CAPACITY REQUIREMENTS:

A. GENERAL: Production and placement capacity shall be sufficient to assure continuous mixing, placing, and finishing of the concrete in each unit of pour during daylight hours. However, the Contractor may be permitted to place concrete at night if adequate lighting facilities are used and prior approval of the proposed operations and facilities is received. It shall be the responsibility of the Contractor to assure the Engineer that an adequate supply of concrete will be furnished and placed to meet the requirements specified below in Sub-Section 500.06.B. Should the Contractor fail to complete a pour for any reason, the Engineer will require the Contractor to form an approved construction joint or to remove the partial pour or to take other remedial measures directed by the Engineer. This shall be done at the Contractor's expense unless the fault lies solely with the Department.

B. MINIMUM RATE OF PLACEMENT: The Contractor shall have enough supervision, manpower, equipment, tools, and materials to assure the proper production, placement, and finish of concrete in each unit of pour in accordance with the minimum requirements for rate of placement specified below. Should the Contractor fail to properly produce, place, and subsequently finish concrete in accordance with these minimum requirements, the Engineer may reject the pour, and will not allow further placement operations of a similar nature and size until corrective measures have been taken which assure the Engineer that minimum placement requirements can be met.

1. BRIDGE SUBSTRUCTURE:

Pour Size, In Cubic Yards	Minimum Rate of Placement, In Cubic Yards Per Hour.
0-25	10
26-50	15
51-75	20
76-100	25
101 and over	30, or as designated on the Plans or in the Special Provisions.

Column placement need not conform to the above rates, but shall meet the requirements of Sub-Section 500.06.B.3.b.

2. BRIDGE SUPERSTRUCTURE

Pour Size, In Cubic Yards	Minimum Rate of Placement, In Cubic Yards Per Hour.
0-25	15
26-50	20
51-75	26
76 and over	30 or as designated on the Plans or in the Special Provisions.

Handrail, parapet, curb, and barrier pours need not conform to the above rates, but they shall be poured at a rate satisfactory to the Engineer.

- Scott Bridge does not like night work and recommends against it except for emergencies. They view that the biggest single problem with night work is safety. They also feel, primarily because of safety, that the time required to do a task at night is about double that during daylight hours.

- To help achieve as-designed deck thicknesses and top bar covers, the Texas DOT "shoots" elevations of the edge girders and/or screed rails as the deck concrete is placed to monitor deflections, and then makes adjustments as necessary. The GADOT annually grades each contractor in achieving the proper <u>deck thickness</u> and <u>rebar cover</u>. Scott Bridge receives a letter each year grading them in these two areas. This motivates the contractors to do a good and proper job in this area.

- Mr. Davis feels that more attention to deck curing would be most helpful in minimizing deck cracking. He feels that currently in Alabama there is probably a wide range in the attention to and quality of deck curing among bridge construction contractors. (Perhaps grading the contractors on quality of curing might be a good idea).

7.2.3 McInnis Corporation. The PI met with Mr. Tim McInnis, Vice President of McInnis Corporation, at his office in Montgomery, AL on September 2, 1999 to discuss the effects of increasing the thickness of bridge decks and other issues related to enhancing the durability and service life of bridge decks. Mr. McInnis' primary observations and comments are listed below.

- There would be very little increase in deck cost associated with an increase in deck thickness of 1". The only additional cost would be that of the additional concrete at approximately $\$60/yd^3$.

- Any standardization in bridge/deck dimensions, detailing, materials, etc. would be very helpful in improving construction quality and in reducing construction costs. This will be even more so in the future as construction company experienced people retire and are replaced by people inexperienced in construction. Using standard component details and procedures will help them achieve a quality product. (It should be noted, that this is similar to the reason that engineers and project managers are pushing for the use of Super-P or self-leveling concrete in Japan, i.e., their inexperienced concrete placing work force can place self-leveling concrete without honeycombing, etc.).

- Standardization of girder clear spacing would probably be of greatest value to the construction contractor. It would allow use of one SIP metal form size and allow greater reuse of diaphragm forms or components.

- They do have difficulty getting 4000 psi concrete with ALDOT's deck concrete mixture when they are not using one of the larger and more experienced ready-mix concrete producers.

- They typically do not have difficulty staying in the 4% - 6% specs on air content.

- When they are working in large municipal areas with experienced and sophisticated ready-mix concrete producers, they would prefer that ALDOT allow them the freedom to use a mixture of their choice (subject to ALDOT's approval). They feel that in these ready-mix producers know best what they can and can not produce in concrete, and giving them more freedom would be good for the contractor and for ALDOT.

- When they are working in rural areas and can only get concrete from a small ready-mix plant with limited experience, then they would look to use an ALDOT standard mixture, but one that provides excessive strength, e.g., that should provide 5000 psi concrete if 4000 psi is what is called for. They feel that it is foolish and poor management to risk having to tear out bridge decks because they are trying to use the cheapest concrete to do the job.

- They believe that it is very helpful to use superplasticizers in bridge deck concrete. They believe that the owner gets a product with less honeycombing, that is better consolidated, and is finished better (easier to finish). However, in hot weather, the rate of slump loss can be a problem, and an experienced concrete producer may be needed in this case.

- They strongly prefer to place bridge deck by pumping. It is much quicker and less "wear and tear" on the workers and thus probably results in a better concrete deck job. The cost of renting a pump truck makes the placing cost somewhat higher, but McInnis views that it is well worth the additional cost. When McInnis places decks by crane-and-bucket, they typically do it with a 2 yd^3 bucket.

- They believe that a, if not "the," big contributor to early bridge deck cracking in recent times is poor quality raw materials. They believe that ALDOT specs regarding concrete raw materials should be looked at closely and that good policing of these specs need to be implemented. For example they believe that dirty/dusty coarse aggregate is often used by batching plants along with cements that are out of specs, and that these lead to inferior concrete with high incidences of cracking.

- They believe that errors in achieving proper deck thickness and rebar cover are primarily the result of errors in estimating the girder dead load deflections. These are typically overestimated and typically results in thinner decks with less top bar cover than specified.

7.3 Closure

Based on detailed discussions with three Alabama bridge construction contractors, the primary conclusions regarding the effects of increasing bridge deck thickness from construction and cost perspectives are as follows:

- A 1" increase in deck thickness would translate into an additional deck cost of approximately $0.20/ft^2 or $1.70/yd^2 of deck.
- A 1" increase in deck thickness would translate into an additional deck DL of 12 psf. The contractors view this as having a negligible effect on the bridge support girders and substructures sizes, construction requirements, and costs.
- Thicker decks would allow contractors more tolerance in achieving proper vertical locations of all deck rebar, and this would reduce deck cost.
- Acknowledging that they are not engineers, the contractors felt that thinner decks and more flexible superstructures did seem to deflect excessively, and that bridges with thinner decks seemed to experience a greater rate of deck deterioration.
- A reasonably rapid rate of concrete placement ($\approx$ 20 yd^3/hour) is important in achieving a quality bridge deck placement in the summer months.
- Use of superplasticizers in the deck concrete would be very helpful in improving construction quality and in reducing construction time and cost.
- Any and all standardization pertaining to bridge decks (e.g., deck concrete, thickness, girder spacing, rebar sizes and spacing, etc.) would be helpful to construction contractors and result in reduced cost and improved product quality. Standardization of the clear spacing between support girders would probably be of greatest help.
- Errors in achieving proper deck thickness and top bar cover are primarily caused by overestimating bridge DL deflections.

From a design perspective, bridge edge girders normally control the sizing of the support girders, and deck overhangs would not necessarily need to be thickened (because they are already thickened) if the deck thickness is increased by 1". Thus, the edge girders would experience very little additional DL and their sizing would probably be uneffected by a 1" increase in deck thickness. If this is the case, the other support girders would not be effected. The 1" decks would provide stiffer decks and superstructure systems, and should provide stronger and more durable systems as well.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

The results of performing a parameter sensitivity study, talking with ALDOT bridge design engineers and bridge contractors, and a survey of other state DOTs all point to increasing bridge deck thickness to a minimum of 8 in. These results are discussed below.

From the parameter sensitivity study several important facts and relationships became evident. All of the parameters investigated, with the exception of two (deck unit weight and initial cost), support increasing Alabama's bridge deck thicknesses to 8". Increasing the deck thickness from 7" to 8" will have the following positive effects. The bridge deck will have an increase in uncracked moment of inertia (by a factor of 1.49), section modulus (1.31), torsional stiffness (1.49), cracked moment of inertia (1.25), fundamental natural frequency (1.14), cracking moment capacity (1.31), ultimate moment capacity (1.19), yield line failure load (1.19), and punching shear force capacity (1.23). The bridge deck will have a decrease in cracking propensity (0.80), crack width (0.77), live load deflection (0.67), fundamental period of vibration (0.89) maximum deck stress (0.77), and arching action membrane force (0.84). Other positive effects will be in the deck/girder system. There will be increases (depending on girder type) in uncracked moment of inertia (1.05-1.07), section modulus (1.07-1.08), and fundamental natural frequency (1.15-1.16). There will be decreases (depending on girder type) in maximum deck stress (0.92-0.94), live load deflection (0.94-0.95) and fundamental period of vibration (0.87). Increasing deck thickness will also help distribute the load laterally to more girders.

The only negative effects of increasing deck thickness from 7" to 8" will be increases in deck unit weight (1.14) and cost. The 1" increase will cause the deck unit weight to increase 6-12 psf depending on whether the concrete is added to the top or underside of the deck. This will increase the cost of the bridge deck around $0.20ft^2, and would translate into an increase in deck/bridge initial cost of around 2%-3%. However, increasing the deck thickness from 7" to 8" would also increase the deck service life (10-50%) which would reduce the life cycle cost of the deck/bridge.

As stated in Chapter 6 bridge design engineers and analytical analyses indicate that thicker decks should perform better and the additional deck cost should be minimal. Support girders would not be affected except on rare occasions. No change in bearings or abutments or bents would be required and only a possible slight change in the foundation. All of these negligible effects can be seen in Chapter 6.

Several important facts and opinions were obtained from discussions with bridge contractors as indicated below.

- A 1" increase in deck thickness would translate into an additional deck cost of approximately \$0.20/ft^2 or \$1.70/yd^2.
- A 1" increase in deck thickness would translate into an additional deck DL of 12 psf. The contractors view this as having a negligible effect on the bridge support girders and substructure sizes, construction requirements, and costs.
- Thicker decks would allow contractors more tolerance in achieving proper vertical locations of all deck rebar, and this would reduce deck cost.
- Any and all standardization pertaining to bridge decks (e.g., deck concrete, thickness, girder spacing, rebar sizes and spacing, etc.) would be helpful to construction contractors and result in reduced cost and improved product quality.
- Errors in achieving proper deck thickness and top bar cover are primarily caused by overestimating bridge DL deflections.

Several important facts and opinions were obtained from the mail survey questionnaire as indicated below. The survey reflects a justification for Alabama to increase its bridges deck thickness, and top bar cover, and to standardized its bridge deck thickness.

- The survey data shows that Alabama has the thinnest bridge decks in the nation.
- The range of deck thicknesses in the U.S. (for typical girder spacings of 6'-9') is 7" (Alabama) to 9.5" (New York), with the mode being 8".
- Most states (55%) employ a top bar cover of 2.5", and (64%) employ a bottom bar cover of 1".
- Most states believe that excessive deck cracking and premature deterioration are related to deck thickness (58%), and are related to superstructure flexibility (64%).
- Most states (65%) believe that it is a good idea to use a "standard" deck thickness (with exceptions allowed) in order to enhance deck quality and durability through greater standardization in construction and inspection, and half of the states (50%) are currently employing this practice.
- Half (5) of the southern states use a deck top bar cover of 2", and the other half (5) use a cover in excess of 2" (around 2.5")
- All of the southern states use thicker decks than Alabama, with the additional thickness varying from about 1/2"-1 1/2".

8.2 Recommendations

From the information developed and collected in this study, the following recommendation can be made concerning bridge deck thickness in Alabama. Bridge deck thickness should be increased from a minimum of 7" to a minimum of 8". This will increase the durability and longevity of the bridge deck and in turn lower the life cycle cost of the deck/bridge. The deck thickness of 8"should become the standard and used throughout the state except for unique circumstances. This will improve construction quality and lower construction cost by simplifying the process. Also the deck top bar cover should be increase from 2" to 2 1/2" to allow for construction errors and errors in overestimating the deck DL deflection. This will help insure a minimum cover of 2". All of these changes will help improve the quality of bridge decks in Alabama with minimal increase in initial cost and an anticipated reduction in life cycle cost.

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Appendix A

Mail Survey Questionnaire on Bridge Deck Thickness

A1. Questionnaire Transmittal Letter

A2. Survey Questionnaire

A3. Summary of Survey Questionnaire Comments

A1. Questionnaire Transmittal Letter

Auburn University

Auburn University, Alabama 36849-5337

College of Engineering

Department of Civil Engineering
Harbert Engineering Center

Telephone: (334) 844-4320
Fax: (334) 844-6290

March 4, 1999

Mr. Steve Bradford
Chief Bridge Engineer
Alaska DOT
3132 Channel Drive
Juneau, AK 99811

Dear Mr. Bradford:

I have prepared the enclosed questionnaire in hopes of learning of your requirements and experiences regarding "Bridge Deck Thickness Requirements and Cost/Benefits of Using Thicker Decks."

The Alabama DOT employs some of the thinnest bridge decks in the country with thicknesses ranging from 61/4" to 7 3/4" up until about a year ago. They have since increased the minimum thickness to 7" and now employ a range of 7" to 7 3/4". Many of their decks in highly-trafficked areas are exhibiting significant deck cracking which is leading to surface spalling and a need for major deck rehabilitation or replacement. Most feel that the thin decks on fairly flexible support girders are the primary cause of the poor service life. Thus, we are working with the Alabama DOT in looking into the cost/benefits (primarily in enhanced service life) of employing thicker decks. This is the impetus for this questionnaire.

Your assistance via completing the enclosed questionnaire would be most helpful to us in investigating this issue. The questionnaire is short and focuses on CIP reinforced concrete bridge deck thicknesses and means to improve deck service life. We hope to assemble and analyze the survey data in April, and therefore, receipt of your response by April 5, 1999 will be appreciated.

Please contact me at the number on this letterhead or on the last page of the questionnaire if you have any questions. Thank you in advance for your help.

Sincerely,

G. E. Ramey
Professor of Civil Engineering

GER/jb

Enclosure

A2. Survey Questionnaire

Survey Questionnaire
on
CIP Reinforced Concrete Bridge
Deck Thickness

Instructions

It is recognized that poor construction practices can negate the best of designs and lead to inferior bridge decks. However, the focus of this questionnaire is on design parameter criteria to enhance the serviceability performance of bridge decks. The questions pertain to required design values for deck thickness and related parameters for CIP reinforced concrete bridge decks. The Alabama Department of Transportation (ALDOT) has a "Bridge Deck Structured Design Criteria" summary figure (see attached Fig. 1) depicting most of the deck design parameter values for different girder spacing. If your DOT has a similar summary figure would you please enclose a copy as part of your response to this questionnaire.

Questions

1. If your DOT has a deck structural design summary figure similar to the attached Fig. 1, will you please share it with us via attaching it to this questionnaire. If you were able to provide a summary figure, proceed to Question 4. If not, proceed to the next question.

2. What deck thickness do you typically use for the following girder spacings?

Girder Spacing	6'	7'	8'	9'	10'
Deck Thickness					

3. What deck top and bottom rebar clearance do you call for? CL_{top} =
Cl_{bottom} =

4. Do you use deicing salt on your decks as needed during the winter season?
☐ Yes ☐ No

5. Do you design-in an extra 1/2" (or other) to the deck thickness (above the top mat) to allow for possible future grinding for rideability? ☐ Yes ☐ No

6. What strength/grade deck concrete and reinforcing steel do you call for?
f'_c =
F_y =

7. Do you require, or typically employ, a wearing surface on your bridge decks?
☐ Yes ☐ No

If yes, what wearing surface material and thickness do you typically use?

8. What LL deflection requirement do you impose for your bridge superstructures?

9. It is reported in the literature that many state DOTs enacted deck design changes of increasing deck thicknesses (typically from 6" - 6½" to 8" - 8½"), and increasing cover on the top mat steel in the 1970s. Did your state make such changes?
☐ Yes ☐ No

If yes, has there been any significant improvements in performance and/or service life due to increasing the deck thickness?

10. Have you made any significant changes (other than those in the previous question if applicable) in your deck thickness and/or rebar cover requirements in the last 30 or so years? ☐ Yes ☐ No

If yes, what were these changes, and how would you assess the results of the changes?

11. Do you believe that excessive deck cracking and premature deterioration are related to deck thickness? ☐ Yes ☐ No

If yes, in what way, and do you have any data or performance information to support your belief?

12. Do you believe that excessive deck cracking and premature deterioration are related to bridge superstructure flexibility? ☐ Yes ☐ No

If yes, in what way, and do you have any data or performance information to support your belief?

13. Do you believe it would be a good idea to use a "standard" (with exceptions allowed) deck thickness (of say 8"), and to vary girder spacing and the reinforcing steel as necessary in order to more closely "standardize" deck construction (and Inspection) practices, and in turn enhance the quality of the as-built decks?

    ☐ Good Idea ☐ Bad Idea

    Why?

14. Please list any design actions you would suggest to enhance the serviceability/service life of CIP reinforced concrete bridge decks.

15. Contact person in the event that additional questions arise.

    Name: ______________________________

    Organization: ______________________________

    Address: ______________________________

    ____

    ____

    Phone: ______________________________

    E-Mail: ______________________________

Thank you for taking the time to share your expertise and experience with us! Please return this questionnaire (along with your Deck Structural Design Criteria figure if you have one) in the self-addressed and stamped envelope provided. In the event that the envelope is lost, please return to:

Dr. G. Ed Ramey
Department of Civil Engineering
Harbert Engineering Center
Auburn University, AL 36849
Telephone: (334) 844-6292
Fax: (334) 844-6290
E-Mail: geramey@eng.auburn.edu

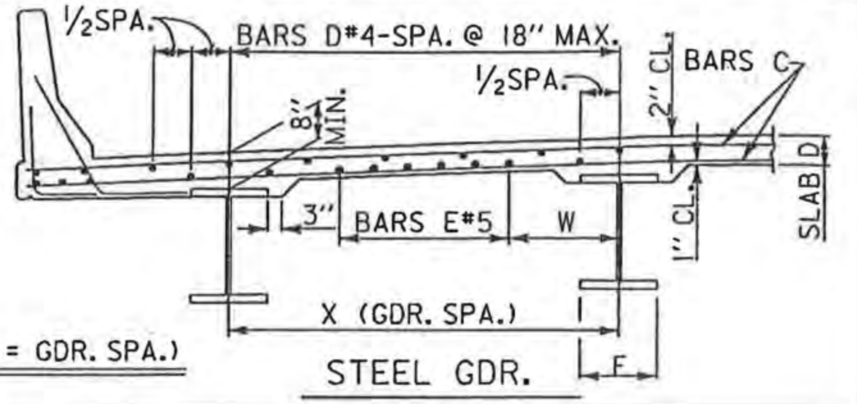
HS 20-44

AUGUST 1990

FOR STEEL REINF. IN BARRIER RAIL, SEE STD. DWG. I-131, SHT. 2 OF 3.

$$W = \frac{(X) - F}{5} + \frac{F}{2} + 2''$$

$$S = (X) - \frac{F}{2} \qquad (X = \text{GDR. SPA.})$$



$$W = \frac{(X) - F}{5} + \frac{F}{2} + 2'' \qquad S = (X) - F$$

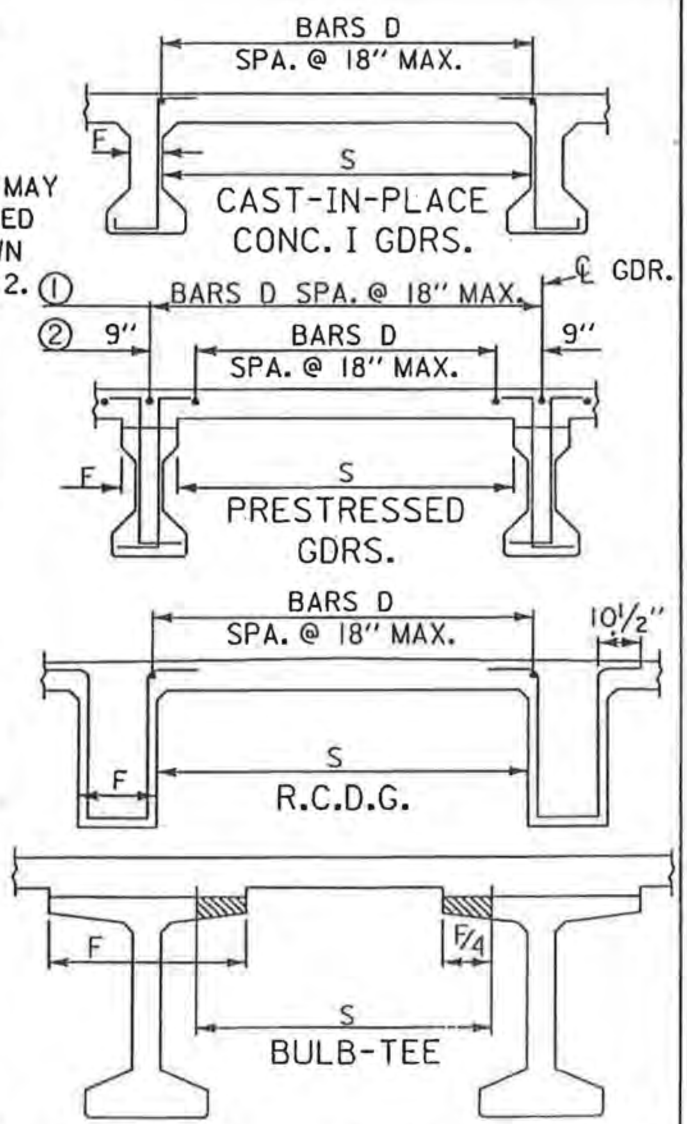
BULB-TEE ONLY

$$W = \frac{S}{5} + \frac{F}{4} + 2'' \qquad S = (X) - \frac{F}{2}$$

USE NEAREST "S" VALUE IF ½ WAY BETWEEN USE HIGHER "S" VALUE.

BARS D MAY BE PLACED AS SHOWN IN 1 OR 2.

S	D	BARS C SIZE @ IN. O.C.		BARS E REQ'D.	A_S
4.0'	7"	#5	6½"	5	.57
4.5'	7"	#5	6½"	5	.57
5.0'	7"	#5	6½"	5	.57
5.5'	7"	#5	6"	5	.62
6.0'	7"	#5	5½"	6	.68
6.5'	7"	#5	5½"	6	.68
7.0'	7"	#5	5½"	7	.68
7.5'	7"	#5	5¼"	8	.71
8.0'	7"	#5	5"	8	.74
8.5'	7¼"	#5	5"	9	.74
9.0'	7½"	#5	4½"	10	.83
9.5'	7½"	#5	4½"	11	.83
10.0'	7¾"	#5	4½"	11	.83



ALABAMA HIGHWAY DEPARTMENT STANDARD BRIDGE SLAB

Figure A.1 ALDOT Bridge Deck Structural Design Criteria

A.3 Summary of Survey Questionnaire Comments

Survey Comments on Deck Cracking and Design Changes:

Listed below is a summary of pertinent comments made by the survey responders to questions on (1) deck cracking and premature deterioration being related to deck thickness and/or superstructure flexibility, (2) changes made in the last 30 years in deck thickness and/or rebar cover requirements, and (3) other design actions recommended to enhance deck service life. The comments were extracted directly from the questionnaire responses, and thus there are some repeats. The repeats were included as they give additional insight and weight to the comments. Two exceptions to this were the comments "to use a high performance concrete (to reduce permeability and chloride ion penetration)", and "to use epoxy coated rebar for corrosion protection." These comments were made numerous times, but were not repeated below because ALDOT's deck problem is not primarily a rebar corrosion problem. The comments are not listed in any order of significance or applicability to Alabama, but rather simply as they were extracted from the survey responses.

- Premature cracking results from bad cures.
- Inadequate cover results in delaminations and spalling.
- Good cover (2½ to 3") and designing to AASHTO leads to relatively thick decks that give good service life.
- We changed concrete strength from 3000 to 4500 psi, top bar cover from 2 to 2½", kept deck thickness as before, limit concrete stress to 1400 psi and rebar stress to 20 ksi, and decks are performing better than those built earlier.
- Reduce the allowable LL deflection as the span length increases.

- Bridge decks constructed in the 1930's and 1940's do not have as much cracking as those recently constructed. The older bridges were constructed with thicker decks.

- Most deck deterioration is caused by intrusions of deicing chlorides. Thin decks develop cracks from excessive flexibility and may have insufficient top rebar cover which facilitate chloride penetration to the rebar.

- Will attain sound decks if they are properly cured. Decks should be wet cured.

- Curing is the No. 1 way to get a good strong crack free deck. Also use 8" thick decks with 2" cover on steel top and bottom.

- Thicker decks seem to perform better. Concrete cover is also very important.

- Thinner decks (5½-6") allow more chloride penetration and result in more corrosion and deterioration.

- Use close spacing on bar supports and tie all steel. Also curing and the method used for deck grooving have a significant influence on durability.

- Improved concrete mixture and cover requirements may be more important than thickness.

- Prior to 1970's, most bridge girders were structural steel and decks were thin. This system was quite flexible and the decks deteriorated rather rapidly. After the 1970's, most girders were concrete with thicker decks and are stiffer and performing very well.

- Recommend increasing deck thickness and using a water and burlap cure only.

- Steel stringer bridges, which are generally more flexible, have greater deck deterioration problems, as do bridges with high volumes of truck traffic.

- Switching to load factor design allowed a more flexible superstructure, and we saw more transverse cracking at this time.

- We believe thin decks have greater cracking problems.

- Decks less than 7" thick appear to be too thin.

- We do not use thin decks because of their great sensitivity to design capacity. General construction tolerance (¼ to ½" variation in plan location) can have a significant effect on deck capacity if thin decks are used.

- We're replacing decks built in the late 60's and early 70's that have thin decks. It appears that we're having to replace more of the thinner decks than thicker decks that were built in the same time period.

- Thin decks are more prone to have inadequate rebar cover which leads to premature deterioration. Also, limit deck overhangs to 4 ft.

- Thinner decks (D < 8") typically have cracked more in our past designs. Most have been serviceable but are not as aesthetically pleasing to the public and cause more maintenance problems.

- Stress improved curing practices and proper top bar cover (we use 2½").

- Use a high performance low permeability concrete, and a good water cure immediately after concrete has been placed.

- Use epoxy coated rebars.

- Increase rebar cover.

- The majority of our deck replacements and structural overlays have been for our 6-6½" thick decks.

- We have noticed more deck cracking as our superstructures have generally become more flexible.

- Indicate locations of deck rebar splices rather than allowing the contractor to chose the locations.

- I do not believe the deck thickness is that critical.

- Amount of cover and the concrete mixture design seem to be the most significant factors to enhance the service life of concrete bridge decks.

- Use 8" as a minimum deck thickness, and improve curing and construction practices.

- Most transverse cracking is due to poor deck curing practices. We recommend a minimum deck thickness of 8" with a 2½" cover on the top bars.

- We had a significant deck cracking problem with our thin deck (6½") on steel stringer bridges in the 1960's.

- The present generation of bridge designers is relying on computers to meet the absolute minimum requirements of the design specifications. Distortional fatigue cracking of steel beams is occurring at an alarming rate. Optimized (terribly flexible) superstructures, wide beam spacing, and lack of experienced designers appear to be the culprits.

- Increase top cover from 2 to 2½" for corrosion protection, and increase the deck thickness to that required for HS25 loading.

Survey Comments on Using a Standard Deck Thickness

Listed below are pertinent comments made in response to the survey question on using a "standard" bridge deck thickness. Recall that in Table 3.1 and Figure 3.1, most of the responders (58%) favored using a "standard" deck thickness to enhance the quality of the as-built deck, while 30% had an unfavorable view of such an action, and 12% were either neutral on the action or did not respond to this question. Also recall in Table 3.2 the large number of states, i.e., approximately 22, that appear to use a "standard" or "minimum/standard" deck thickness. The comments below have been separated into those favorable to using a "standard" deck thickness and those opposed.

Favorable Comments:

- It may be good to "standardize" the deck thickness to eliminate other construction related problems.

- Use 8" deck thickness as a minimum. This will probably work on most bridges, but go to thicker decks for wide girder spacing.

- Prevents designers from skimping on deck thickness. An 8" thickness is about the minimum to assure adequate cover and minimum spacing between top and bottom mats.

- This has been our practice for the most part with relatively good success.

- This is basically what we do; however, we have several standard deck thicknesses based on beam spacing.
- This is what we do and we are satisfied with the results.
- We do this to some extent in our design charts.
- By standardizing deck construction, the variabilities and errors that can arise are seemingly reduced, particularly with the labor force that constructs them.
- Eliminates a variable in slab design, allows us to use standard panel design, and bid history for decks may be more consistent.
- Leads to consistency with the contractors.
- The less variation in the field, the better the construction will be, which should enhance quality.
- Improve quality through standardization.
- Simplifies construction and construction details. Easy to vary rebar and rebar spacing. Consistent unit cost per square foot for all decks.
- We feel the benefits of having an 8" deck (having the room for two layers of rebar with 2½" top bar cover) out-weigh any savings we would get from using a thinner deck.
- We have already implemented this. Using "LRFD" Empirical Deck Design has also standardized the reinforcing. Repetition in construction should increase quality.
- We try to standardize our design, but I do not think you can go to only one deck thickness.
- This seems to be what our state is doing with 7½" thick decks. Standardization where feasible is always a good idea.
- Our practice is to use a minimum deck thickness of 8" with 2½" top bar cover. Reinforcement and/or thickness beyond the minimum are used to accommodate wider girder spacings.
- We have standardized our bridge deck thickness at 8" and standardized our deck steel spacing as well. Repetition makes things cheaper and is conducive to less errors during constructions.

- We have a minimum deck thickness of 8", and increase this thickness for wider girder spacings.

Unfavorable Comments:

- Some decks would be overdesigned and others might be underdesigned. There would not be an increase in stiffness of the deck as the spacing increased. Greater girder spacing would produce increased concrete stresses and more flexibility.

- Unnecessary-using a 2½ to 3" top steel cover and designing to AASHTO will give good performance and still allow flexibility in design.

- Deck thickness should be optimized - girders are quite expensive.

- It would be good to have only a minimum deck thickness.

- A good standardization would be the beam spacing. Span of falsework is more critical than 1" or 2" in deck thickness.

- A poor system would be a 8" thick deck with a lot of rebar due to a wide beam spacing.

- Our job is to "engineer" the best solution, not just default to a conservative albeit safe design.

- Material usage not economized. Sometimes you can use thinner decks, and sometimes you may need thicker decks.

- We use a minimum deck thickness of 8½" and still experience cracking. There seems to be variables other than deck thickness and structure flexibility which cause cracking problems, e.g., concrete mixture design and curing.

- Not economical, better that deck thickness is proportional to beam spacing with a standardized minimum thickness.

- Use 8" as a minimum deck thickness.

- Girder spacing should not be controlled by the deck, but by the overall economy of the combined section or by site constraints.

Appendix B

Deck Support Girder Design Examples

B. DECK SUPPORT GIRDER DESIGN EXAMPLES

EXAMPLE B1 - Noncomposite Rolled Steel Girder Bridge

Problem Statement. Design the simple span noncomposite rolled steel girder bridge of Figure B.1 with 10.5-mm span for a *HL*-93 live load. Roadway width is 13 420-mm curb to curb. Allow for a future wearing surface of 75-mm thick bituminous overlay. Use f'_c =30 MPa and M270 Gr345 steel. Design in compliance with AASHTO (1994) LRFD Bridge Specifications. This example was extracted from Ref. ().

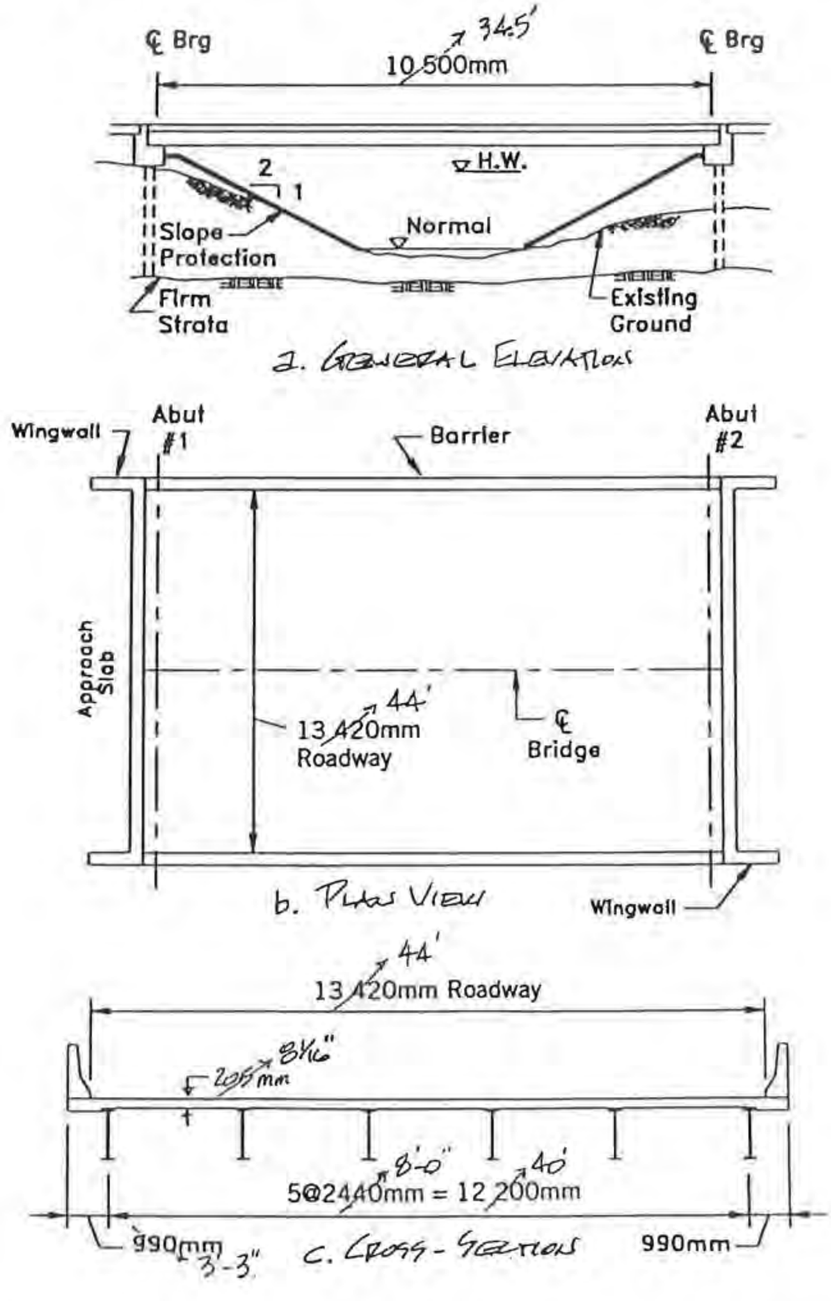


Fig. B.1 Noncomposite rolled steel girder bridge design example ().

For a uniformly distributed load, w,

$$M_{cen} = M_{\max} = \frac{wL^2}{8} = \frac{w(10.5)^2}{8} = 13.78w$$

$$V_{END} = V_{MAX} = \frac{wL}{2} = \frac{w(10.5)}{2} = 5.25w$$

DEAD LOAD ANALYSIS:

Assume a girder weight of 1.5 kN/m.

1. *Interior Girders*

DC	Deck Slab	$(2400)(10^{-9})(9.81)(205)(2440) = 11.78$ kN/m
	Girder	$= \underline{1.5 \text{ kN/m}}$
		$w_{DC} = 13.28$ kN/m

DW 75-mm bituminous paving = $(2250)(9.81)(10^{-9})(75)(2440)$

$w_{DW} = 4.04$ kN/m

2. *Exterior Girders*. Using deck design results for reaction on exterior girder,

DC	Deck Slab	4.60 kN/m
	Overhang	6.75 kN/m
	Barrier	6.74 kN/m
	Girder	$\underline{1.5 \text{ kN/m}}$
		$w_{DC} = 19.59$ kN/m

DW 75-mm bituminous paving

$$w_{DW} = (2250)(10^{-9})(9.81)(75)(610 + \frac{2440}{2})$$

$= 3.03$ kN/m

LIVE LOAD MOMENT ANALYSIS:

See Fig. B.2 below

$$M_{LL+IM} = mg\left[\left(M_{Truck}\ or\ M_{Tandom}\right)\left(1 + \frac{IM}{100}\right) + M_{Lane}\right]$$

↑ Distribution Factor

$$M_{Truck} = 145(2.625) + (145 + 35)(0.475) = 466\ kN\ m$$

$$M_{Tandom} = 110(2.625 + 2.025) = 512\ kN\ m \quad \leftarrow governs$$

$$M_{Lane} = \frac{9.3(10.5)^2}{8} = 128\ kN\ m$$

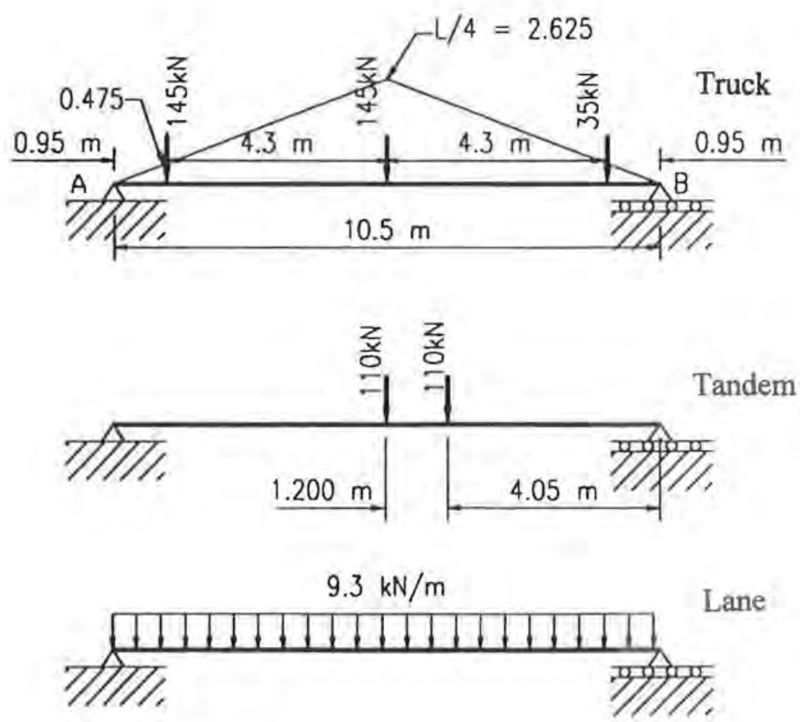


Fig. B.2 Truck, tandem, and lane load placement for maximum moment at Location 105().

Therefore,

Interior Girders

$$M_{LL=IM} = 0.748[512(1.33) + 128] = 605\ kN\ m$$

Exterior Girders

$$M_{LL+IM} = 0.762[512(1.33) + 128] = 616\ kN\ m$$

Shear does not control (which is typical) and is not shown.

Unfactored maximum moments and shears for interior and exterior girders are shown in Table B.1 below.

TABLE B.1. Unfactored Maximum Moments and Shears

Girder	Load Type	$w(kN/m)$	Moment (kN m) M_{105}	Shear (kN) V_{100}
Interior	*DC*	13.28	183	70
	DW	4.04	56	21
	LL + IM	N/A	605	301
Exterior	*DC*	19.59	270	103
	DW	3.03	42	16
	LL + IM	N/A	616	278

GIRDER SECTION REQUIRED:

Strength I limit State

a. Interior girder

Factored shear and moment

$$U = \eta[1.25DC + 1.50DW + 1.75(LL + IM)]$$

$$V_u = 0.95[1.25(70) + 1.50(21) + 1.75(301)] = 613 \text{ kN}$$

$$M_u = 0.95[1.25(183) + 1.50(56) + 1.75(605)] = 1302 \text{ kN m}$$

b. Exterior girder

Factored shear and moment

$$V_u = 0.95[1.25(103) + 1.50(16) + 1.75(278)] = 607 \text{ kN}$$

$$M_u = 0.95[1.25(270) + 1.50(42) + 1.75(616)] = 1405 \text{ kN m} \quad \leftarrow \text{controls}$$

Note that the exterior girder controls the girder size/design for this example.

Required Plastic Section Modulus, Z

$$\varphi_f M_n \geq M_u, \ \varphi_f = 1.0, M_n = M_p = Z F_y$$

$$Z F_y \geq M_u$$

Assuming compression flange is fully braced and section is compact,

$$req'd\ Z \geq \frac{M_u}{F_y} = \frac{1405 \ x \ 10^6}{345} = 4.072 \ x \ 10^6 \ mm^3$$

Try W760 x 134, $Z = 4.63 \times 10^6$ mm^3, $S = 4000 \times 10^3$ mm^3, $I = 1500 \times 10^6$ mm^4,

$$w_g = (134)(9.81)(10^{-3})$$

$= 1.31$ kN/m $\leftarrow$ Less than the assumed value of 1.5, but close. Therefore, O.K.

Note, W760 x 134 → W30 x 90

Shear capacity checks out for this girder section, as do service load requirements.

EXAMPLE B1a - Design Example B1 Modified for a 1" Thicker Deck

Problem Statement. Let us now examine the effect of increasing the deck thickness 1" on the girder design for the bridge of Example B1. Example B1 is repeated below with a 1" thicker deck to make this assessment.

Assume the deck thickness is increased from 205 mm to 230mm

Therefore,

$$w_{DC}^{\text{INTERIOR}} = \begin{array}{rl} 13.21 & \text{Deck} \\ \underline{1.5} & \text{Girder (Assumed)} \\ 14.78 & \text{kN/m} \end{array}$$

$$w_{DC}^{\text{EXTERIOR}} = \begin{array}{rl} 5.16 & \text{Deck} \\ 7.57 & \text{Overhang} \\ 6.74 & \text{Barrier} \\ \underline{1.5} & \text{Girder (Assumed)} \\ 20.97 & \text{kN/m} \end{array}$$

Therefore, the dead loads, moments, and shears in the Table B1 would be altered as indicated in Table B.2.

TABLE B.2 Unfactored Maximum Moments and Shears for Thickened (230mm) Deck

Girder	Load Type	$w(kN/m)$	Moment (kN m) M_{105}	Shear (kN) V_{100}
Interior	DC	~~13.28~~ 14.78	~~183~~ 204	~~70~~ 78
	DW	4.04	56	21
	$LL + IM$	N/A	605	301
Exterior	DC	~~19.28~~ 20.97	~~270~~ 289	~~103~~ 110
	DW	3.03	42	16
	$LL + IM$	N/A	616	278

Therefore, the new Strength I Limit State design values for the girders are as follows.

Interior Girder:

$$V_u = 0.95\ [1.25\ (78) + 1.50\ (21) + 1.75\ (301)]$$

$$= 623 \text{ kN}$$

$$M_u = 0.95\ [1.25\ (204) + 1.50\ (56) + 1.75\ (605)]$$

$$= 1328 \text{ kNm}$$

Exterior Girder:

$$V_u = 0.95\ [1.25\ (110) + 1.50\ (16) + 1.75\ (278)]$$

$$= 616 \text{ kN}$$

$$M_u = 0.95\ [1.25\ (289) + 1.50\ (42) + 1.75\ (616)]$$

$$= 1427 \text{ kNm} \qquad \leftarrow \text{Controls}$$

Therefore,

$$Z_{REQD} \geq \frac{M_u}{F_y} = \frac{1427 \times 10^6}{345} = 4.137 \times 10^6 mm^3$$

Thus, again use a

$$W760{\times}134 \quad (Z = 4.63{\times}10^6 mm^3)$$

as before.

Note that increasing deck thickness by 1", caused the controlling girder ultimate moment (factored moment) to increase from

$$M_u = 1405 \quad \xrightarrow{\text{to}} \quad 1427 \text{ kNm}$$

This is only a 1.57% increase in design moment, and as can be seen above, does not affect the final girder sizing/design.

It should be noted that Service Load State requirements should be checked as well. In this regard, live load deflections, which are the only deflections checked, are decreased by increasing the deck thickness. Thus this will check-out as satisfactory. Girder fatigue stress conditions will be slightly worsened due to a slightly increased (assuming no unintentional composite action behavior) mean stress level due to the dead load of the additional 1" of deck thickness. The fatigue stress range due to LL will remain unchanged. Since the original girder design was no where near fatigue controlling the design, this was not rechecked for the case of increasing the deck thickness.

This example assumes noncomposite behavior and thus no stiffening or strengthening of the system accrues from increasing the deck thickness (only the negative effect of increasing the DL). If the bridge is designed for composite behavior, then increasing the deck thickness would indeed stiffen and strengthen the superstructure and should result in the same girder design (as the one with the thinner deck), or possibly a smaller/lighter girder design.

Also, this example employs rolled steel sections for the bridge girders. If prestressed concrete girders were used, they would be substantially heavier. Thus the increased dead load moment from increasing the deck thickness would be the same as in the example above; however, percentage wise, the increase in DL moment would be substantially smaller than in this example. Also, the increase in total M_u (percentage wise) would be smaller than in this example. Thus, an increase in total M_u of perhaps 1% would probably be a reasonable estimate for increasing the deck thickness by 1", if prestressed girders are used as the superstructure support elements. In turn, this should translate into being able to use the same prestressed girders as in the thinner deck design.

Last, it should be noted that noted that this example case is for a fairly short bridge (≈35'). For longer bridges the effect of increasing the deck thickness may have a greater effect. The $\Delta w^{DL}{}_{DECK}$ will remain constant independent of bridge span, but the $\Delta\ w^{DL}{}_{girder}$ will increase with increasing span length, and the DL moments vary as the square of the span length. An example utilizing a longer span (31.5 m or 100 ft) is checked, in Example B5.

EXAMPLE B2 - Composite Rolled Steel Beam Bridge

Problem Statement. Design the simple span composite rolled steel beam bridge of Figure B.3 with 10.5-m span for a *HL*-93 live load. Roadway width is 13 420 mm curb to curb. Allow for a future wearing surface of 75-mm thick bituminous overlay. Use f'_c = 30 Mpa and M270 Gr250 steel. Design in compliance with AASHTO (1994) Bridge Specifications. Note that this is the same bridge as in Example B1 except for being composite and using Gr250 steel. This example was extracted from Ref.()

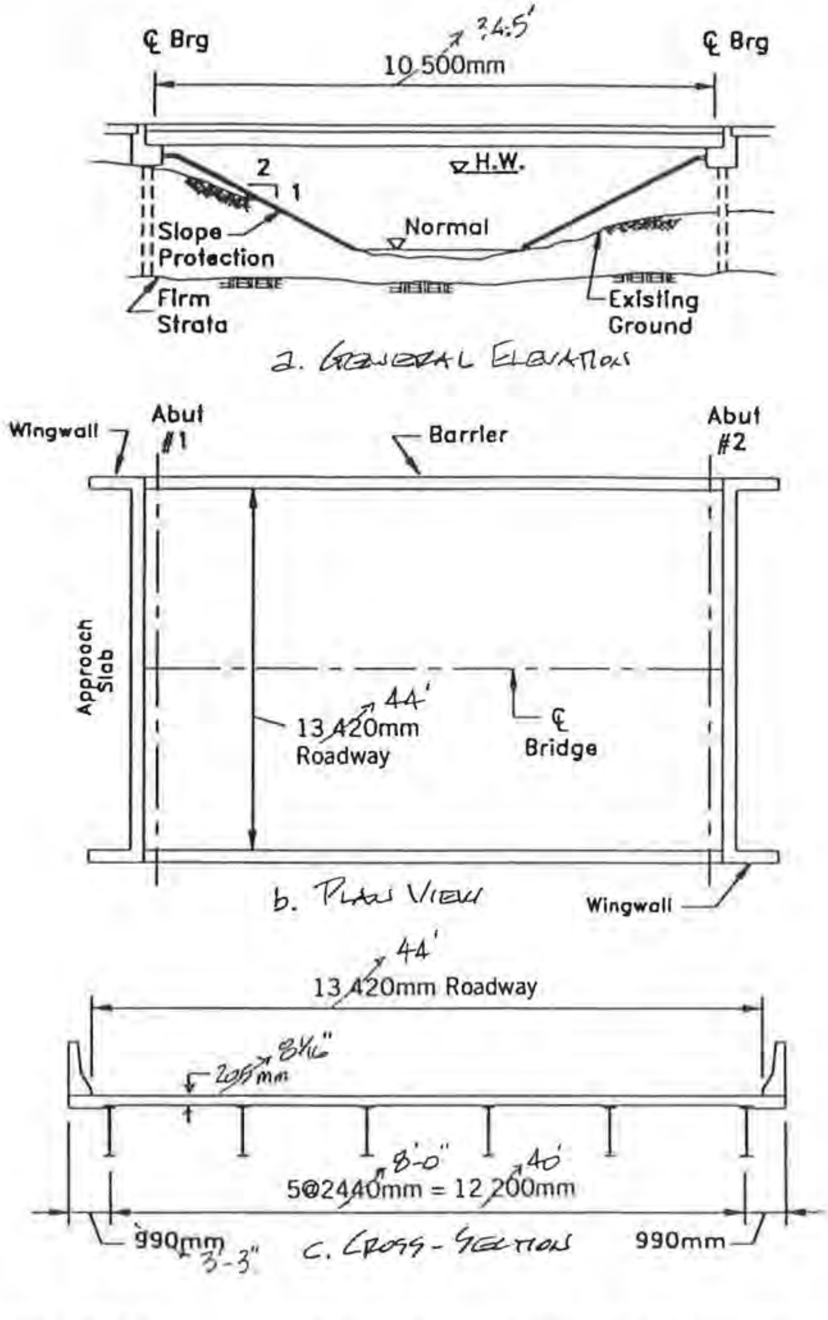


Fig. B.3 Composite rolled steel beam bridge design example ()

Calculate Force Effects from Dead and Live Loads

$D1$ = dead load of structural components and their attachments, acting on the noncomposite section

$D2$ = future wearing surface

$D3$ = barriers that have a cross-sectional area of 197 312 mm^2

A 50-mm x 300-mm average concrete haunch at each girder is used to account for camber and unshored construction.

Assume a girder weight of 1.5 kN/m.

From Example B1, for a uniformly distributed load, w,

$$M_{105} = 13.78w$$

$$V_{100} = 5.25w$$

1. Interior Girders

$D1$ Deck slab $(2400)(10^{-9})(9.81)(205)(2440) = 11.78$ kN/m

Girder $= 1.5$ kN/m

Haunch $(2400)(10^{-9})(300)(50) = \underline{0.35 \text{ kN/m}}$

$w^I_{D1} = 13.63$ kN/m

$D2$ 75 mm bituminous paving

$$w^I_{D2} = (2250)(9.81)(10^{-9})(75)(2440) = 4.04\ kN/m$$

$D3$ Barriers, one-sixth share

$$w^I_{D3} = \frac{2(197\ 312)(2400)(9.81)}{6(10^9)} = 1.55\ kN/m$$

2. Exterior Girders

$D1$ Deck slab $(2400)(10^{-9})(9.81)$
$\times [(230)(990) + (205)(1220)] = 11.25$ kN/m

Girder $= 1.5$ kN/m

Haunch $= \underline{0.35 \text{ kN/m}}$

$w^{E}_{D1} = 13.10$ kN/m

$D2$ 75-mm bituminous paving

$$w_{D2}^{E} = (2250)(9.81)(10^{-9})(75)(1220 + 990 - 380) = 3.03 kN/m$$

$D3$ Barriers, one-sixth share

$w^{E}_{D3} = 1.55\ kN/m$

Table B.3 summarized the unfactored moments and shears at critical sections for interior and exterior girders. The values for LL + IM are the same as those in Example B1.

TABLE B.3 Interior and Exterior Girder Unfactored Moments and Shears

Girder	Load Type	$w(kN/m)$	Moment (kN m) M_{105}	Shear (kN) V_{100}
Interior	$D1$	13.63	188	72
	$D2$	4.04	56	21
	$D3$	1.55	21	8
	$LL + IM$	N/A	605	301
Exterior	$D1$	13.10	181	69
	$D2$	3.03	42	16
	$D3$	1.55	21	8
	$LL + IM$	N/A	616	278

Design Required Sections

1. Strength Limit State
   a. Interior beam - factored shear and moment

$$U = \eta[1.25DI + 1.50D2 + 1.25D3 + 1.75(LL + IM)]$$

$$V_u = 0.95[1.25(72) + 1.50(21) + 1.25(8) + 1.75(301)] = 625\ kN$$

$$M_u = 0.95[1.25(188) + 1.50(56) + 1.25(21) + 1.75(605)] = 1334 kNm \quad \leftarrow Controls$$

   b. Exterior beam - factored shear and moment

$$V_u = 0.95[1.25(69) + 1.50(16) + 1.25(8) + 1.75(278)] = 576\ kN$$

$$M_u = 0.95[1.25(181) + 1.50(42) + 1.25(21) + 1.75(616)] = 1324\ kNm$$

2. Consider Loading and Concrete Placement Sequence

Case 1	Weight of girder and slab (*D*1). Supported by steel girder alone.
Case 2	Superimposed dead load (*FWS*, curbs, and railings) (*D*2 and *D*3). Supported by long-term composite section.
Case 3	Live load plus impact (*LL* + *IM*). Supported by short-term composite section.

3. Determine Effective Flange Width

For interior girders the effective flange width is the least of

a. One-quarter of the average span length

b. Twelve times the average thickness of the slab, plus the greater of the web thickness or one-half of the width of the top flange of the girder

c. Average spacing of adjacent girders

Assume the girder top flange is 200 mm wide

$$b_i = min \left| \begin{array}{l} (0.25)(10\ 500) = 2625\ mm \\ (12)(190) + \dfrac{200}{2} = 2380\ mm \\ 2440\ mm \end{array} \right.$$

Therefore, $b_i = 2380\ mm$

For exterior girders the effective flange width is one-half the effective flange width of the adjacent interior girder, plus the least of

a. One-eighth of the effective span length

b. Six times the average thickness of the slab, plus the greater of one-half of the web thickness or one-quarter of the width of the top flange of the girder.

c. The width of the overhang

$$b_e = \frac{b_i}{2} + min \left| \begin{array}{l} (0.125)(10\ 500) = 1313\ mm \\ (6)(190) + \dfrac{200}{4} = 1190\ mm \\ 990\ mm \end{array} \right.$$

Assume 15 mm of 205 mm is sacrificial for future grinding

$$Therefore,\ b_e = \frac{b_i}{2} + 990 = \frac{2380}{2} + 990 = 2180\ mm$$

4. Modular Ratio

For $f_c' = 30\ MPa,\ n = 8$

5. Cover Plates

For economy, the lightest and shallowest beam with the largest cover plate possible gives the best design. The length of the cover plate, L_{cp}, must satisfy

$$L_{cp} \geq 2d_s + 900$$

where d_s = depth of the steel section (mm)

6. Trial Section Properties
   a. Steel section at midspan

Try W610 x 101 with 10 mm x 200-mm cover plate

Properties of W610 x 101 are taken from AISC (1992). The steel section is shown in Fig. B.4 and calculations for the steel section properties are summarized in Table B.4.

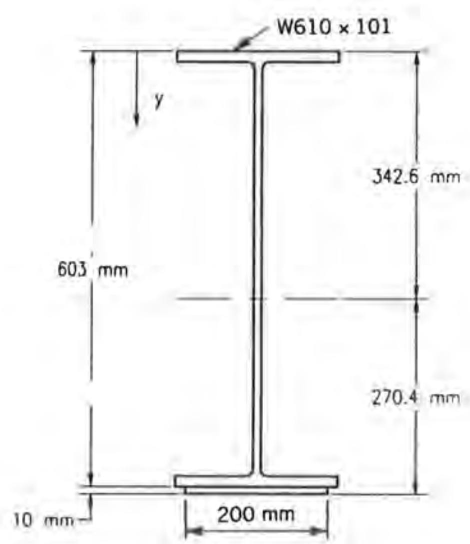


Fig. B.4 Steel section at midspan.

TABLE B.4 Steel Section Properties

Component	A	y	Ay	$y - \bar{y}$	$A(y - \bar{y})^2$	I_o
Beam	*12 900*	$\frac{603}{2} = 301.5$	*3.889 x 10⁶*	*-41.4*	*21.8 x 10⁶*	*764 x 10⁶*
Cover Plate	*2000*	*603 + 5 = 608*	*1.216 x 10⁶*	*265.4*	*140.9 x 10⁶*	*17 x 10³*
Σ	*14 900*		*5.105 x 10⁶*		*162.7 x 10⁶*	*764 x 10⁶*

$$I_x = (162.7\ x\ 10^6) + (764\ x\ 10^6) = 926.7\ x\ 10^6\ mm^4$$

$$\bar{y} = \frac{\Sigma Ay}{\Sigma A} = \frac{5.105\ x\ 10^6}{14900} = 342.6\ mm,\ y_t = -342.6\ mm$$

$$y_b = 613 - 342.6 = 270.4\ mm$$

$$S_t = \frac{I_x}{y_t} = \frac{926.7\ x\ 10^6}{-342.6} = -2.705\ x\ 10^6\ mm^3$$

$$S_b = \frac{I_x}{y_b} = \frac{926.7\ x\ 10^6}{270.4} = 3.427\ x\ 10^6\ mm^3$$

(Note: positive y is downward from centroid of section.)

b. Composite section, $n = 8$, at midspan. Figure B.5 shows the composite section with a haunch of 25 mm, a net slab thickness (without 15 mm sacrificial wearing surface) of 190 mm, and an effective width of 2380 mm. The composite section properties calculations are summarized in Table B.5

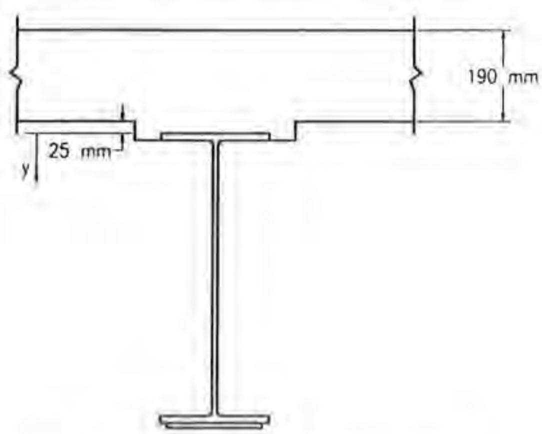


Fig. B.5 Composite section at midspan.

TABLE B.5 Short-Term Composite Section Properties, n = 8, b_i = 2,380

Component	A	y	Ay	$y - \bar{y}$	$A(y - \bar{y})^2$	I_o
Concrete (b_i x t_s/n)	56 525		-6.783 x 10^6	-96.5	526 x 10^6	170 x 10^6
Steel	14 900	342.6	5.105 x 10^6	366.1	1997 x 10^6	927 x 10^6
Σ	71 425		-1.678 x 10^6		2523 x 10^6	1097 x 10^6

*b_i is used because interior girders control the moment design.

$$I_x = (2523\ x\ 10^6) + (1097\ x\ 10^6) = 3620\ x\ 10^6\ mm^4$$

$$\bar{y} = \frac{\Sigma Ay}{\Sigma A} = \frac{-1.678\ x\ 10^6}{71425} = -23.5\ mm,\ y_t = 23.5\ mm$$

$$y_b = 613 + 23.5 = 636.5\ mm$$

$$S_t = \frac{3620\ x\ 10^6}{23.5} = 154.0\ x\ 10^6\ mm^3$$

$$S_b = \frac{3620\ x\ 10^6}{636.5} = 5.69\ x\ 10^6\ mm^3$$

c. Composite section, $3n$ = 24, at midspan. The composite section properties calculations, reduced for the effect of the creep in the concrete slab, are summarized in Table B.6.

TABLE B.6 Long-Term Composite Section Properties, 3n = 24.

Component	*A*	*y*	*Ay*	$y - \bar{y}$	$A(y - \bar{y})^2$	I_o
*Concrete** b_i x $t_s/3n$)	*18 842*	*-120*	*-2.261 x* 10^6	*-204.3*	*786 x* 10^6	*56.7 x* 10^6
Steel	*14 900*	*342.6*	*5.105 x* 10^6	*258.3*	*994 x* 10^6	*926.7 x* 10^6
Σ	*33 742*		*2.844 x* 10^6		*1780 x* 10^6	*983.4 x* 10^6

**b_i is used because interior girders control the moment design.*

$$I_x = (1780 \; x \; 10^6) + (983.4 \; x \; 10^6) = 2763.4 \; x \; 10^6 \; mm^4$$

$$\bar{y} = \frac{\Sigma Ay}{\Sigma A} = \frac{2.844 \; x \; 10^6}{33742} = 84.3 \; mm, \; y_t = -84.3 \; mm$$

$$y_b = 613 - 84.3 = 528.7 \; mm$$

$$S_t = \frac{2763.4 \; x \; 10^6}{-84.3} = -32.78 \; x \; 10^6 \; mm^3$$

$$S_b = \frac{2763.4 \; x \; 10^6}{528.7} = 5.227 \; x \; 10^6 \; mm^3$$

7. ***Stresses.*** Stresses in top and bottom of girder for strength limit state are given in Tables B.7 and B.8. Yielding has occurred in the bottom flange. The implications of this are discussed in subsection 8 below.

TABLE B.7 Compressive Stresses in Top of Steel Beam due to Factored Loading (Interior Girder)

Load	M_{D1}	M_{D2}	M_{D3}	M_{LL+IM}	S_t Steel	S_t Composite	Stress (MPa)
D1	235				-2.705 x 10^6		-86.9
D2		84				-32.78 x 10^6	-2.6
D3			26			-32.78 x 10^6	-0.8
LL +IM				1059		154.0 x 10^6	6.9
Total							-83.4
						$\eta = 0.95$	-79.2

TABLE B.8 Tensile Stresses in Bottom of Steel Beam due to Factored Loading (Interior Girder)

Load	M_{D1}	M_{D2}	M_{D3}	M_{LL+IM}	S_b Steel	S_b Composite	Stress (MPa)
D1	235				3.427×10^6		68.6
D2		84				5.227×10^6	16.1
D3			26			5.227×10^6	5.0
LL +IM				1059		5.69×10^6	186.1
Total							275.8
						$\eta = 0.95$	262.0

8. Others

Because the bottom flange elastic stress (262 MB) for the factored loads/moments exceed the yield stress (250 MPa), the section must satisfy certain requirements, which upon checking it does. The section was checked for member proportions, fatigue, compactness, and plastic moment capacity and all of the requirements for flexure were satisfied. Shear, construction, deflection and other service load conditions were also checked and were satisfactory for the girder section.

The design of the composite, simple span, rolled steel beam bridge is summarized in Figure B.6.

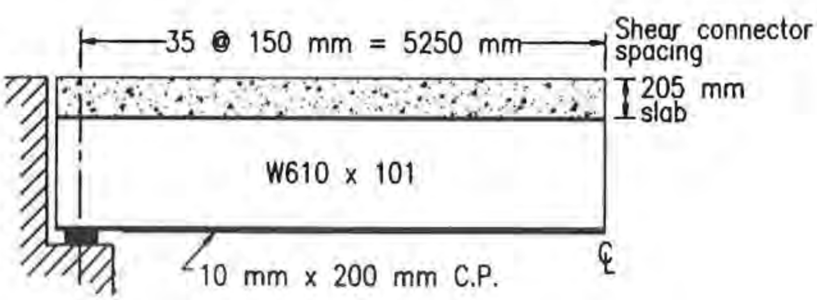


Fig. B.6 Design sketch of composite rolled steel girder.

EXAMPLE B2a - Design Example B2 Modified for a 1" Thicker Deck

Problem Statement. Determine the effect of increasing the deck thickness 1" on the girder design for the bridge of Example B2. Example B2 is repeated below with a 1" thicker deck to make this assessment, i.e., assume the deck thickness in Example B2 is increased from 205 mm to 230 mm.

Calculate Force Effects from Dead and Live Loads

As in Example B2,

$$M_{105} = 13.78w$$

$$V_{100} = 5.25w$$

Dead load and unfactored design moments and shears are shown below for Example B2 corrected for the 1" increase in deck thickness and deck DL.

1. *Interior Girders*

*D*1	Deck slab	$(2400)(10^{-9})(9.81)(\cancel{205}\ 230)(2440)$	=	$\cancel{11.78}$ 13.21 kN/m
	Girder		=	1.5 kN/m
	Haunch	$(2400)(10^{-9})(9.81)(300)(50)$	=	0.35 kN/m
			w^I_{DI} =	$\cancel{13.63}$ 15.06 kN/m

*D*2 75 mm bituminous paving

$$w^I_{D2} = (2250)(9.81)(10^{-9})(75)(2440) = 4.04\ kN/m$$

*D*3 Barriers, one-sixth share

$$w^I_{D3} = \frac{2(197\ 312)(2400)(9.81)}{6(10^9)} = 1.55\ kN/m$$

2. *Exterior Girders*

$D1$	Deck Slab	$(2400)(10^{-9})(9.81)$		12.62
		X [(~~230~~ 255)(990) + (~~205~~ 230)(1220)]	=	~~11.25~~ kN/m
	Girder		=	1.5 kN/m
	Haunch		=	0.35 kN/m
		w^E_{D1}	=	~~13.10~~ kN/m
				14.47

$D2$ 75-mm bituminous paving

$$w^E_{D2} = (2250)(9.81)(10^{-9})(75)(1220 + 990 - 380) = 3.03\ kN/m$$

$D3$ Barriers, one-sixth share

$w^E_{D3} = 1.55$ kN/m

Table B.9 summarizes the unfactored moments and shears at critical sections for interior and exterior girders. The values for D2, D3, and LL + IM are the same as those in Example B2.

TABLE B.9 Interior and Exterior Girder Unfactored Moments and Shears

Girder	Load Type	*w(kN/m)*	Moment (kN m) M_{105}	Shear (kN) V_{100}
Interior	$D1$	~~13.63~~ 15.06	~~188~~ 208	~~72~~ 79
	$D2$	4.04	56	21
	$D3$	1.55	21	8
	$LL + IM$	N/A	605	301
Exterior	$D1$	~~13.10~~ 14.47	~~181~~ 199	~~69~~ 76
	$D2$	3.03	42	16
	$D3$	1.55	21	8
	$LL + IM$	N/A	616	278

Design Required Sections

1. *Strength Limit State*
    a. Interior beam - factored shear and moment

$$U = \eta[1.25D1 + 1.50D2 + 1.25D3 + 1.75(LL + IM)]$$

$$V_u = 0.95[1.25(\overset{79}{\cancel{72}}) + 1.50(21) + 1.25(8) + 1.75(301)] = \overset{633}{\cancel{625}}\ kN$$

$$M_u = 0.95[1.25(\overset{208}{\cancel{188}}) + 1.50(56) + 1.25(21) + 1.75(605)] = \overset{1359}{\cancel{1334}}\ kNm$$

    b. Exterior beam - factored shear and moment

$$V_u = 0.95[1.25(\overset{76}{\cancel{69}}) + 1.50(16) + 1.25(8) + 1.75(278)] = \overset{585}{\cancel{576}}\ kN$$

$$M_u = 0.95[1.25(\overset{199}{\cancel{181}}) + 1.50(42)\ 1.25(21) + 1.75(616)] = \overset{1347}{\cancel{1324}}\ kNm$$

2. *Consider Loading and Concrete Placement Sequence*

Case 1 Weight of girder and slab (D1). Supported by steel girder alone.

Case 2 Superimposed dead load (FWS, curbs, and railings) (D2 and D3). Supported by long-term composite section.

Case 3 Love load plus impact (LL+IM). Supported by short-term composite section.

3. *Determine Effective Flange Width*
    For interior girders the effective flange width is the least of
    a. One-quarter of the average span length
    b. Twelve times the average thickness of the slab, plus the greater of the web thickness or one half the width of the top flange of the girder
    c. Average spacing of adjacent girders
    Assume the girder top flange is 200 mm wide

$$b_i = min \left| \begin{array}{l} (0.25)(10\ 500) = 2625\ mm \\ (12)(\overset{215}{\cancel{190}}) + \dfrac{200}{2} = \overset{2680}{\cancel{2380}}\ mm \\ 2440\ mm \end{array} \right.$$

Assume 15 mm of 230 mm is sacrificial for future grinding

Therefore $b_i = \overset{2440}{\cancel{2380}}$ mm

For exterior girders the effective flange width is one-half the effective flange width of the adjacent interior girder, plus the least of

a. One-eighth of the effective span length
b. Six times the average thickness of the slab, plus the greater of one-half of the web thickness or one-quarter of the width of the top flange of the girder
c. The width of the overhang

$$b_e = \frac{b_i}{2} + min \left| \begin{array}{l} (0.125)(10500) = 1313\ mm \\ (6)(\overset{215}{\cancel{190}}) + \dfrac{200}{4} = \overset{1340}{\cancel{1190}}\ mm \\ 990\ mm \end{array} \right.$$

Therefore $$b_e = \frac{b_i}{2} + 990 = \frac{\overset{2440}{\cancel{2380}}}{2} + 990 = \overset{2210}{\cancel{2180}}\ mm$$

4. *Modular Ratio*

For $f_c' = 30\ MPa,\ n = 8$

5. *Cover Plates*

For economy, the lightest and shallowest beam with the largest cover plate possible gives the best design. The length of the cover plate, L_{cp}, must satisfy

$$L_{cp} \geq 2d_s + 900$$

where d_s = depth of the steel section (mm)

6. *Trial Section Properties*
a. Steel section at midspan

Try W610 X 101 with 10 mm X 200-mm cover plate

Note, this is the same steel section used in Example B2. See Fig B.4 and Table B.4 (in Example B2) for the trial steel section properties.

b. Composite interior section, $n = 8$, at midspan. Figure B.7 shows the composite section with a haunch of 25 mm, a net slab thickness (without 15 mm sacrificial wearing surface) of 205 mm, and an effective width of 2440 mm. The composite section properties calculations are summarized in Table B.10.

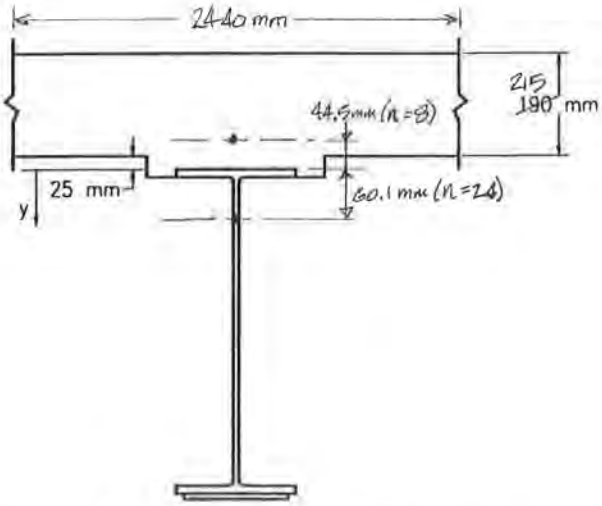


Fig. B.7 Composite section at midspan.

TABLE B.10 Short-Term Composite Section Properties, $n = 8$, $b_i = 2440$ mm

Component	A	y	Ay	$y - \bar{y}$	$A(y - \bar{y})^2$	I_o
Concrete* ($b \times t_s/n$)	65 575	$-25 - \frac{215}{2} = -132.5$	-8.689×10^6	-88.0	508×10^6	253×10^6
Steel	14 900	342.6	5.105×10^6	387.1	2233×10^6	926.7×10^6
Σ	80 475		-3.584×10^6		2741×10^6	1180×10^6

*b_i is used because interior girders control the moment design.

$$I_x = (2741 \ x \ 10^6) + (1180 \ x \ 10^6) = 3921 \ x \ 10^6 \ mm^4$$

$$\bar{y} = \frac{\Sigma Ay}{\Sigma A} = \frac{-3.584 \ x \ 10^6}{80475} = -44.5 \ mm, \ y_t = 44.5 \ mm$$

$$y_b = 613 + 44.5 = 657.5 \ mm$$

$$S_t = \frac{3921 \ x \ 10^6}{44.5} = 88.1 \ x \ 10^6 \ mm^3$$

$$S_b = \frac{3921 \ x \ 10^6}{657.5} = 5.96 \ x \ 10^6 \ mm^3$$

(Note: positive y is downward from centroid of section.)

c. Composite section, $3n = 24$, at midspan. The composite section properties calculations, reduced for the effect of creep in the concrete slab, are summarized in Table B.11.

TABLE B.11 Long-Term Composite Section Properties, 3n = 24.

Component	A	y	Ay	$y - \bar{y}$	$A(y - \bar{y})^2$	I_o
Concrete* ($b \ x \ t_s/3n$)	21 858	-132.5	-2.896×10^6	-192.6	811×10^6	84.2×10
Steel	14 900	342.6	5.105×10^6	282.5	1189×10^6	927×10^6
Σ	36 758		2.209×10^6		2000×10^6	1011.2×10^6

*b_i is used because interior girders control the moment design.

$$I_x = (2000 \ x \ 10^6) + (1011.2 \ x \ 10^6) = 3011.2 \ x \ 10^6 \ mm^4$$

$$\bar{y} = \frac{\Sigma Ay}{\Sigma A} = \frac{2.209 \ x \ 10^6}{36758} = 60.1 \ mm, \ y_t = -60.1 \ mm$$

$$y_b = 613 - 60.1 = 552.9 \ mm$$

$$S_t = \frac{3011.2 \ x \ 10^6}{-60.1} = -50.10 \ x \ 10^6 \ mm^3$$

$$S_b = \frac{3011.2 \ x \ 10^6}{552.9} = 5.446 \ x \ 10^6 \ mm^3$$

7. *Stresses.* Stresses in top and bottom of girder for strength limit state are given in Tables B.12 and B.13. Yielding has occurred in the bottom flange. The implications of this are discussed in subsection 8 below.

TABLE B.12 Compressive Stresses in Top of Steel Beam due to Factored Loading (Interior Girder)

Load	M_{D1}	M_{D2}	M_{D3}	M_{LL+IM}	S_t Steel	S_t Composite	Stress (MPa)
D1	260				-2.705×10^6		-96.1
D2		84				-50.10×10^6	-1.7
D3			26			-50.10×10^6	-0.5
LL +IM				1059		88.1×10^6	12.0
Total							-86.3
						$\eta = 0.95$	-82.0

TABLE B.13 Tensile Stresses in Bottom of Steel Beam due to Factored Loading (Interior Girder)

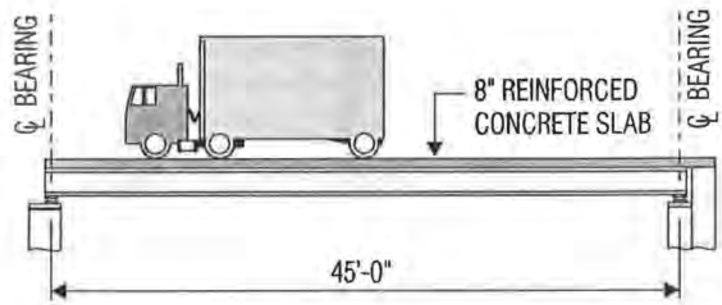
Load	M_{D1}	M_{D2}	M_{D3}	M_{LL+IM}	S_b Steel	S_b Composite	Stress (MPa)
D1	260				3.427×10^6		75.9
D2		84				5.446×10^6	15.4
D3			26			5.446×10^6	4.8
LL +IM				1059		5.96×10^6	177.7
Total							273.8
						$\eta = 0.95$	260.1

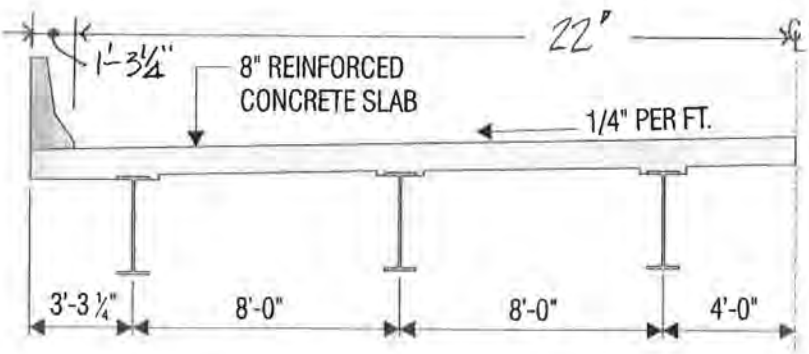
8. Other

   As in Example B2, all the requirements for flexure are satisfied in this design (see subsection 8 in Example B2). Increasing the deck thickness 25 mm (1") from 205 mm to 230 mm did not affect the girder design for the bridge. Thus the final design would be the same as that shown in Fig. B.6 except that the deck thickness would be 230 mm.

Example B3 - Design of a Composite Steel-Concrete Interior Girder

Problem Statement: Design an interior girder for the bridge shown below. Use working stress design (WSD). This example was extracted from Ref. ( ).





GIVEN:

- Bridge cross-section as shown above:
- Span length of 45 ft centerline to centerline of bearings.
- Average haunch depth of 2 in.
- Unshored construction.
- Account for 25 psf future wearing surface.
- Span is simply supported.
- Overpass is a major highway with ADTT of 3,000.

STEP 1: Compute the Effective Flange Width

The effective flange width is defined as:

MINIMUM OF		
1/4 x Span Length = (0.25)(45.00)	=	11.25 ft
Center-to-Center Between Girders	=	8.00 ft
12 x Min. Slab Thickness = (12)(8)(1 ft/12 in)	=	8.00 ft

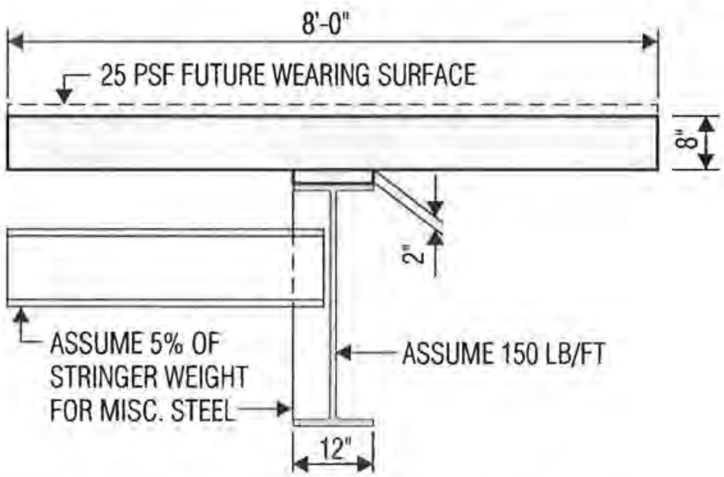
$\Rightarrow b_{eff} =$ 8.0 ft

STEP 2: Compute the Dead Load on Girders
The dead load is composed of the following items:

DL_{slab} = (b_{eff})(slab thickness)(W_{conc})
= (8.0 ft)(8.0 in)(1 ft / 12 in)(0.150 k/ft^3) = 0.800 k/ft

DL_{haunch} = (haunch width)(haunch thickness)(W_{conc})
= (1.0 ft)(2.0 in)(1 ft / 12 in)(0.150 K/ft^3) = 0.025 k/ft

Dl_{steel} = (assumed girder weight) + (misc. steel)
= (0.150 k/ft) + (5%)(0.150 k/ft)(1/100) = 0.157 k/ft
$\Rightarrow$DL = 0.982 k/ft

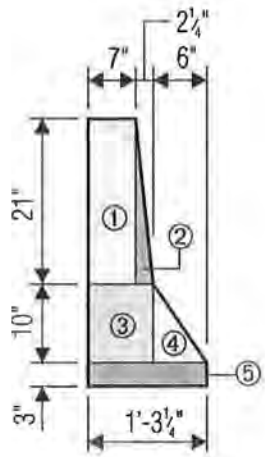


STEP 3: Compute the Superimposed Dead Load on Girder.

Account for future wearing surface:

$$SDL_{ws} = \frac{(width\ of\ roadway)(future\ wearing\ surface)}{number\ of\ girders}$$

$$= \frac{(44\ ft)(0.025\ k/ft^2)}{6\ girders} = 0.183\ \text{k/ft}$$



Calculate weight of parapet by first computing the area of its cross section:

$A_1 = (7\text{in})(21)$	=	147.00 in^2
$A_2 = 1/2(2.25\text{ in})(21\text{ in})$	=	23.62 in^2
$A_3 = (9.25\text{ in})(10\text{ in})$	=	92.50 in^2
$A_4 = 1/2(6\text{ in})(10\text{ in})$	=	30.00 in^2
$A_5 = (15.25\text{ in})(3\text{ in})$	=	45.75 in^2
A_p	=	338.87 in^2

$W_p = (338.87\text{ in}^2)(1\text{ ft}^2/144\text{ in}^2)(0.150\text{ k/ft}^3)$
$= 0.35\text{ k/ft}$

There are two parapets on the bridge which are distributed over all six stringers:

$$SDL_p = 2\ Parapets \cdot \frac{W_p}{No.\ Stringers} = 2 \cdot \frac{0.35\ k/ft}{6\ Stringers} = 0.117\ k/ft$$

$SDL = SDL_{ws} + SDL_p = 0.183\text{ k/ft} + 0.117\text{ k/ft}$

➠SDL = 0.300 k/ft

STEP 4: Compute the Transformed Width of Slab

For $f_c' = 4500$ psi, use: $n = 8$

For live loads and dead loads acting on the girder only ($k = 1$):

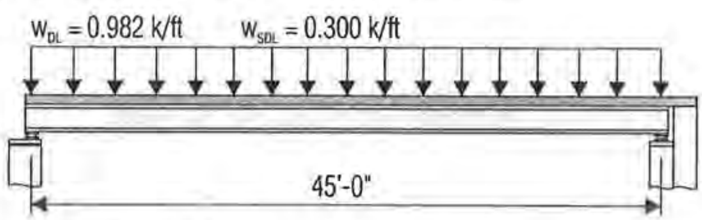
$$b_{tr1} = \frac{b_{eff}}{k \cdot n} = \frac{8.0\ ft}{(1)(8)} = 1.0\ ft \qquad ➠b_{tr1} = 12\ in$$

For superimposed dead loads ($k = 3$):

$$b_{tr3} = \frac{b_{eff}}{k \cdot n} = \frac{8.0\ ft}{(3)(8)} = 0.33\ ft \qquad ➠b_{tr3} = 4\ in$$

Note, k is a modulus of elasticity reduction factor due to creep, i.e., $E_{effective}$ for long duration loads is E_c / k.

STEP 5: Compute Dead & Superimposed Dead Load Moments:



Dead Load Moment:

$$M_{DL} = \frac{wL^2}{8} = \frac{(0.982\ k/ft)(45.00\ ft)^2}{8} \qquad ➠M_{DL} = 248.57\ ft\text{-}k$$

Superimposed Dead Load Moment:

$$M_{SDL} = \frac{wL^2}{8} = = \frac{(0.300\ k/ft)(45.00\ ft)^2}{8} \qquad ➠M_{SDL} = 75.94\ ft\text{-}k$$

STEP 6: Compute Live Load Distribution and Impact Factors:

The wheel load distribution factor for,

- Concrete Floor
- Two or More Traffic Lanes
- On Steel I-Beam Stringers

is,

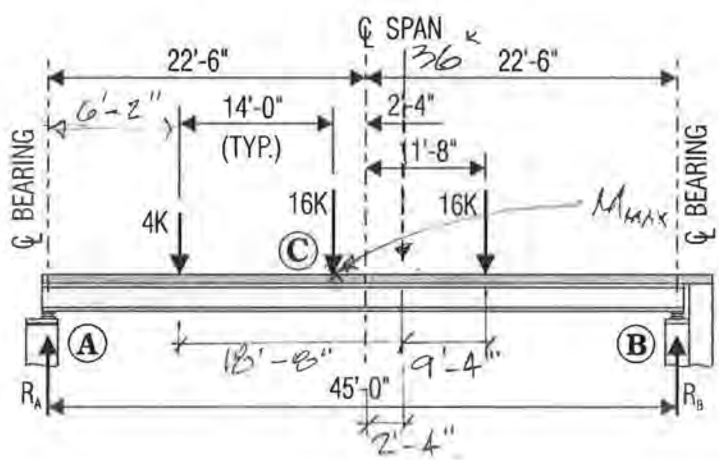
$$DF = \frac{S}{5.5} = \frac{8.0\ ft}{5.5} = 1.45 \qquad \Rightarrow Use\ DF = 1.45$$

The impact factor is,

$$I = \frac{50}{L + 125} = \frac{50}{45\ ft + 125} = 0.29 \qquad \Rightarrow Use\ I = 1.29$$

STEP 7: Compute Live Load Plus Impact Moment:

For maximum LL moment, the HS20-44 truck is located as shown below:



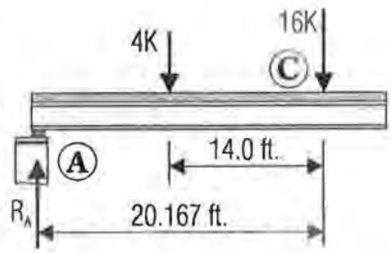
First, solve for the reactions by summing moments about Point A:
$\circlearrowright \Sigma M_A = 0$:

$$(4\ k \cdot 6.167\ ft) + (16k \cdot 20.167\ ft) + (16k \cdot 34.167\ ft) - (R_B \cdot 45\ ft) = 0$$

$$R_B = \frac{894\ ft \cdot k}{45\ ft} = 19.867k \quad so,\ R_A = 36\ k - 19.867\ k = 16.133\ k$$

Now, compute the maximum live load moment:

$M_{LL} = M_{MAX} = (R_A \cdot 20.167ft) - (4k \cdot 14ft) = 269.35 \text{ ft} \cdot k$



Apply the impact and wheel load distribution factors:

$$M_{LL+I} = M_{LL} \cdot DF \cdot I$$
$$M_{LL+I} = (269.35 \text{ ft} \cdot k)(1.45)(1.29)$$
$$\Rightarrow M_{LL+I} = 503.83 \text{ ft-k}$$

STEP 8: Choose a Preliminary Section:

Required steel section (with the compression flange (i.e., top flange) adequately braced by the concrete slab:

$$S_S = \frac{M_{DL}}{F_b} = \frac{(248.57 \ ft{\cdot}k)(12 \ in/ft)}{20 \ k/in^2} = 149.14 \ in^3$$

Required composite section modulus:

$$S_{tr} = \frac{M_{DL} + M_{SDL} + M_{LL+I}}{F_b}$$

$$= \frac{(248.57 + 75.94 + 503.83) \ ft \cdot k \ (12 \ in/ft)}{20 \ k/in^2} = 497.00 \ in^3$$

Area of transformed concrete slab:

$A_{ctr} = b_{tr1} \cdot t = (12 \text{ in})(8 \text{ in}) = 96 \text{ in}^2$

Distance to centroid of slab from top of steel stringer:

$$Y_2 = \frac{t}{2} + Haunch \ Depth = \frac{8 \ in}{2} + 2 \ in = 6 \ in$$

Enter into the AISC Composite Beam Selection Table with:

$S_{tr} = 497 \text{ in}^3$, $A_{ctr} = 100 \text{ in}^2$ (round up for table), and $Y_2 = 6$ in

We will choose as our preliminary section:

$\Rightarrow$ W 33 x 130

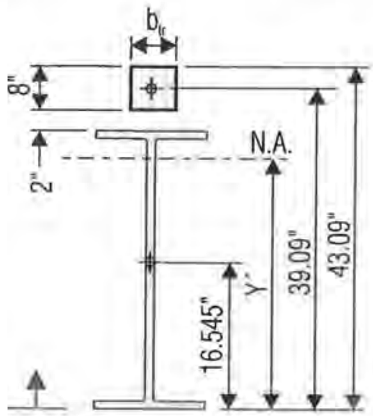
Since, for this section and $Y_2 = 6$ in:

$S_{tr} = 633 \text{ in}^3 > 497 \text{ in}^3$, $A_{ctr} = 100 \text{ in}^2 <$ max $A_{ctr} = 106 \text{ in}^2$, and $S_S = 406 \text{ in}^3 > 149 \text{ in}^3$

STEP 9: Compute Moments of Inertia:

a. Moment of inertia resisting dead load only. Use steel section only:
From AISC Manual of Steel Construction for a W 33 x 130 stringer: ➠I_{DL} = 6710 in^4 and Y'_{DL} = 16.545 in

b. Moment of inertia resisting live load plus impact:



The area and moment of interia of the transformed slab are computed using k= 1. See Step 4, for computation of the transformed width b_{tr1}.

$$A_{ctr1} = b_{tr1}t = (12\ in)(8\ in) = 96\ in^2$$

$$I_{tr1} = \frac{bh^3}{12} = \frac{(12\ in)(8\ in)^3}{12} = 512\ in^4$$

ELEMENT	A (in^2)	Y (in)	AY (in^3)	AY^2 (in^4)	I_0 (in^4)
W 33x130	38.3	16.545	633.673	10484.128	6710.00
SLAB (k=1)	96.0	39.090	3752.640	146690.698	512.00
TOTALS	134.3		4386.313	157174.826	7222.00

$I_z = \Sigma I_0 + \Sigma AY^2 = 7222.0\ in^4 + 157174.826\ in^4 = 164396.826$

$$Y' = \frac{\Sigma AY}{\Sigma A} = \frac{4386.313\ in\ 3}{134.3\ in^2} = 32.66\ in$$

$I_{LL+I} = I_z - (\Sigma A)(Y')^2 = 164396.826\ in^4 - \left(134.3\ in^2\right)(32.66\ in)^2$

➠I_{LL+I} = 21142 in^4 and Y'_{LL+I} = 32.66 in

c. Moment of inertia resisting superimposed dead loads (k = 3):

$$A_{ctr3} = b_{tr3}t = (4in)(8in) = 32\ in^2 \qquad I_{tr3} = \frac{(4\ in)(8\ in)^3}{12} = 170.67\ in^4$$

ELEMENT	A (in^2)	Y (in)	AY (in^3)	AY^2 (in^4)	I_0 (in^4)
W 33x130	38.3	16.545	633.673	10484.128	6710.00
SLAB (k=3)	32.0	39.090	1250.880	4886.899	170.67
TOTALS	70.3		1884.553	59381.027	6880.67

$I_z = \Sigma I_0 + \Sigma AY^2 = 6880.67\ in^4 + 59381.027\ in^4 = 66261.697\ in^4$

$$Y' = \frac{\Sigma AY}{\Sigma A} = \frac{1884.553\ in^3}{70.3\ in^2} = 26.807\ in$$

$$I = I_z - (\Sigma A)(Y')^2 = 66261.697\ in^4 - (70.3\ in^2)(26.807\ in)^2$$

$$\Rightarrow I_{SDL} = 15742\ in^4\ and\ Y'_{SDL} = 26.807\ in$$

STEP 10: Compute and Check Stresses:

a. Stress at bottom fiber of steel section: $f_{bot} = f_{DL} + f_{LL+I} + f_{SDL} < F_b$

$$f_{DL} = \frac{M_{DL}c}{I_{DL}} = \frac{(248.57\ ft \cdot k)(12\ in/ft)(16.545\ in)}{6710\ in^4} = 7.355\ ksi$$

$$f_{LL+I} = \frac{M_{LL+I}c}{I_{LL+I}} = \frac{(503.83\ ft \cdot K)(12\ in/ft)(32.66\ in)}{21142\ in^4} = 9.340\ ksi$$

$$f_{SDL} = \frac{M_{SDL}c}{I_{SDL}} = \frac{(75.94\ ft \cdot k)(12\ in/ft)(26.807\ in)}{15742\ in^4} = 1.552\ ksi$$

$$f_{bot} = 7.355\ ksi + 9.340\ ksi + 1.552\ ksi = 18.25\ ksi < 20\ ksi \quad \text{O.K.}$$

b. Stress at top fiber of steel section: $f_{top} = f_{DL} + f_{LL+I} + f_{SDL} < F_b$

$$f_{DL} = \frac{M_{DL}c}{I_{DL}} = \frac{(248.57\ ft \cdot k)(12\ in/ft)(33.09 - 16.545)in}{6710\ in^4}$$

$$= 7.355\ ksi$$

$$f_{LL+I} = \frac{M_{LL+I}c}{I_{LL+I}} = \frac{(503.83\ ft \cdot k)(12\ in/ft)(33.09 - 32.66)in}{21142\ in^4}$$

$$= 0.123\ ksi$$

$$f_{SDL} = \frac{M_{SDL}c}{I_{SDL}} = \frac{(75.94\ ft \cdot k)(12\ in/ft)(33.09 - 26.807)in}{15742\ in^4}$$

EXAMPLE B3a: Design Example B3 Modified for a 1" Thicker Deck.

Problem Statement: Same problem statement as Example B3, but with a deck thickness of 9" rather 8".

Given: Same as Example B3 except that the deck thickness is 9" rather than 8".

STEP 1: Compute the Effective Flange Width

The effective flange width is defined as:

MINIMUM OF

1/4 x Span Length = (0.25)(45.00)	=	11.25 ft
Center-to-Center Between Girders	=	8.00 ft
12 x Min. Slab Thickness = (12)(~~8~~ 9 in) (1 ft/12 in)	=	~~8.00 ft~~ → 9.00 ft.

➠ b_{eff} = 8.0 ft

STEP 2: Compute the Dead Load on Girders
The dead load is composed of the following items:

DL_{slab} = (b_{eff}) (slab thickness) (w_{conc})
= (8.0 ft) (~~8.0~~ 9.0 in) (1 ft / 12 in) (0.150 k/ft^3) = ~~0.800 k/ft~~ → 0.900 k/ft.

DL_{haunch} = (haunch width) (haunch thickness) (w_{conc})
= (1.0 ft) (2.0 in) (1 ft / 12 in) (0.150 k/ft^3) = 0.025 k/ft

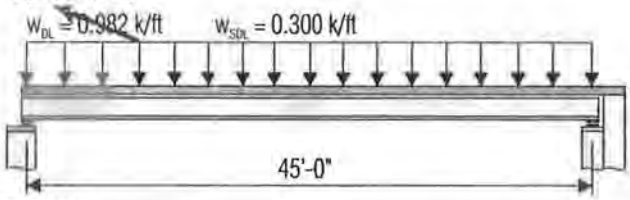
DL_{steel} = (assumed stringer weight + (misc. steel)
= (0.150 k/ft) + (5%) (0.150 k/ft) (1/100) = <u>0.157 k/ft</u>
➠DL = ~~0.982 k/ft~~ → 1.082 k/ft

STEP 3: Compute the Superimposed Dead Load on Stringer
(No Change)

STEP 4: Compute the Transformed Width of Slab
(No Change)

STEP 5: Compute Dead & Superimposed Dead Load Moments:

$W_{DL} = 1.082$ k/ft



Dead Load Moment:

$$M_{DL} = \frac{wL^2}{8} = \frac{(\overset{1.082}{\cancel{0.982}}\ k/ft)(45.00\ ft)^2}{8} \Rightarrow M_{DL} = \cancel{248.57\ ft\text{-}k} \rightarrow 273.88\ ft\text{-}k$$

Superimposed Dead Load Moment:

$$M_{SDL} = \frac{wL^2}{8} = \frac{(0.300\ k/ft)(45.00\ ft)^2}{8} \Rightarrow M_{SDL} = 75.94\ ft\text{-}k$$

STEP 6: Compute Live Load Distribution and Impact Factors:
(No Change)

STEP 7: Compute Live Load Plus Impact Moment:
(No Change)

STEP 8: Choose a Preliminary Section:

Required steel section with the compression flange (i.e., top flange) adequately braced by the concrete slab:

$$S_S = \frac{M_{DL}}{F_b} = \frac{(\overset{273.88}{\cancel{248.57}}\ ft \cdot k)(12\ in/ft)}{20\ k/in^2} = \cancel{149.14\ in^3} \rightarrow 164.33\ in^3$$

Required composite section modulus:

$$S_{tr} = \frac{(M_{DL} + M_{SDL} + M_{LL+I})}{F_b}$$

$$= \frac{(\overset{273.88}{\cancel{248.57}} + 75.94 + 503.83) ft \cdot k(12\ in/ft)}{20\ k/in^2} = \cancel{497.00\ in^3} \rightarrow 512.19\ in^3$$

Area of transformed concrete slab:

$$A_{ctr} = b_{trl} \cdot t = (12\ in)(\cancel{8}^{\,9}\ in) = 96\ in^2 \quad 108\ in^2$$

Distance to centroid of slab from top of steel girder:

$$Y_2 = \frac{t}{2} + Haunch\ Depth = \frac{\cancel{8}^{\,9}\ in}{2} + 2 = 6\ in \quad 6.5\ in$$

Enter into the AISC Composite Beam Selection Table with:

$S_{tr} = 497\ in^3$, $A_{ctr} = 100\ in^2$ *(round up for table), and* $Y_2 = 6\ in$

We will choose as our preliminary section:

➠W 33 x 130

Since, for this section and $Y_2 = 6$ in

$S_{tr} = 633\ in^3 > 497\ in^3$, $A_{ctr} = 100\ in^2$ (108) $\approx$ max $A_{ctr} = 106\ in^2$, and
$S_s = 406\ in^3 > 149\ in^3$ $164\ in^3$

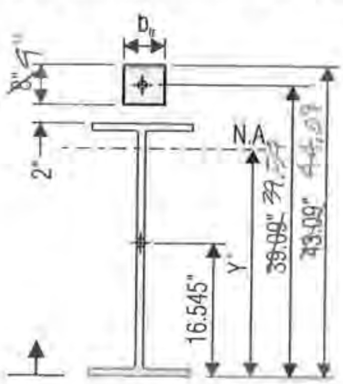
STEP 9: Compute Moments of Inertia:

a. Moment of inertia resisting dead load only. Use steel section only:
From AISC Manual of Steel Construction ➠$I_{DL} = 6710\ in^4$ and
$Y'_{DL} = 16.545$ in

as before → No change

For the composite steel girder/concrete deck section, the increase in deck thickness to 9" substantially changes the composite section properties. The new properties are shown below (rather than showing the old properties corrected).

b. Moment of inertia resisting live load plus impact:



The area and moment of inertia of the transformed slab are computed using k = 1. See Step 4, for computation of the transformed width b_{trl}.

$$A_{ctrl} = b_{trl} t = (12\ in)(9\ in) = 108\ in^2$$

$$I_{trl} = \frac{bh^3}{12} = (12\ in)(9\ in)^3 = 729\ in^4$$

ELEMENT	A (in^2)	Y (in)	AY (in^3)	AY^2 (in^4)	I_0 (in^4)
W 33x130	38.3	16.545	633.673	10484.1281	6710.00
SLAB (k=1)	108	39.59	4275.77	169,275.75	729.00
TOTALS	146.3		4909.39	179759.88	7439.00

$$I_z = \Sigma I_0 + \Sigma AY^2 = 7429.00\ in^4 + 179,759.88\ in^4 = 187,198.88\ in^4$$

$$Y' = \frac{\Sigma AY}{\Sigma A} = \frac{4909.39\ in^3}{146.3\ in^2} = 33.56\ in$$

$$I_{LL+I} = I_z - (\Sigma A)(Y')^2 = 187,198.88\ in^4 - (146.3\ in^2)(33.56\ in)^2$$

$$\Rightarrow I_{LL+I} = 22,425\ in^4\ and\ Y'_{LL+I} = 33.56\ in$$

c. Moment of inertia resisting superimposed dead loads (k = 3):

$$A_{ctr3} = b_{tr3}t = (4in)(9\ in) = 36\ in^2 \quad I_{tr3} = \frac{(4\ in)(9\ in)^3}{12} = 243\ in^4$$

ELEMENT	A (in^2)	Y (in)	AY (in^3)	AY^2 (in^4)	I_0 (in^4)
W 33 x 130	38.3	16.545	633.673	10484.128	6710.00
SLAB (k=3)	36.0	39.59	1425.24	56425.25	243.0
TOTALS	74.3		2058.91	66,909.38	6953.0

$$I_Z = \Sigma I_0 + \Sigma AY^2 = 6953.00\ in^4 + 66,909.38\ in^4 = 73,862.38\ in^4$$

$$Y' = \frac{\Sigma\ AY^2}{\Sigma\ A} = \frac{2058.91\ in^3}{74.3\ in^2} = 27.71\ in$$

$$I = I_Z - (\Sigma A)(Y'^2) = 73,862.38\ in^4 - (74.3\ in^2)(27.71\ in)^2$$

$$\Rightarrow I_{SDL} = 16,812\ in^4\ and\ Y'_{SDL} = 27.71\ in$$

STEP 10: Compute and Check Stresses:

Because of the changes in section properties, all stresses were changed. Thus, as in **Step 9**, the new stresses are shown below rather than the old stresses with changes.

a. Stress at bottom fiber of steel section: $f_{bot} = f_{DL} + f_{LL+I} + f_{SDL} < F_b$

$$f_{DL} = \frac{M_{DL}c}{I_{DL}} = \frac{(273.88\ ft{\cdot}k)(12\ in/ft)(16.545\ in)}{6710\ in^4} = 8.104\ ksi$$

$$f_{LL+I} = \frac{M_{LL+I}c}{I_{LL+I}} = \frac{(503.83\ ft{\cdot}k)(12\ in/ft)(33.56\ in)}{22{,}425\ in^4} = 9.048\ ksi$$

$$f_{SDL} = \frac{M_{SDL}c}{I_{SDL}} = \frac{(75.94\ ft{\cdot}k)(12\ in/ft)(27.71\ in)}{16{,}812\ in^4} = 1.502\ ski$$

$$f_{bot} = 8.104\ ksi + 9.048\ ksi + 1.502\ ksi = 18.65 < 20\ ksi \qquad O.K.$$

b. Stress at top fiber of steel section: $f_{top} = f_{DL} + f_{LL+I} + f_{SDL} < F_b$

$$f_{DL} = \frac{M_{DL}c}{I_{DL}} = \frac{(273.88\ ft{\cdot}k)(12\ in/ft)(33.09 - 16.545)\ in}{6710\ in^4} = 8.104\ ksi$$

$$f_{LL+I} = \frac{M_{LL+I}c}{I_{LL+I}} = \frac{(503.83\ ft{\cdot}k)(12\ in/ft)(33.09 - 33.56)\ in}{22{,}425\ in^4} = -\ 0.127\ ksi$$

$$f_{SDL} = \frac{M_{SDL}c}{I_{SDL}} = \frac{(75.94\ ft{\cdot}k)(12\ in/ft)(33.09 - 27.71)\ in}{16{,}812\ in^4} = 0.292\ ksi$$

$$f_{top} = 8.104\ ksi - 0.127\ ksi + 0.292\ ksi = 8.27\ ksi < 20\ ksi \qquad O.K.$$

c. Stress at top fiber of concrete: $f_c = f_{LL+I} + f_{SDL} < F_c$

$$f_{LL+I} = \frac{M_{LL+I}c}{k{\cdot}n{\cdot}I_{LL+I}} = \frac{(503.83\ ft{\cdot}k)(12\ in/ft)(44.09 - 33.56)\ in}{(1)(8)(22{,}425\ in^4)} = 0.355\ \text{ksi}$$

$$f_{SDL} = \frac{M_{SDL}c}{k{\cdot}n{\cdot}I_{SDL}} = \frac{(75.94\ ft{\cdot}k)(12\ in/ft)(44.09 - 27.71)\ in}{(3)(8)(16{,}812\ in^4)} = 0.037\ \text{ksi}$$

$f_c = 0.355\ ksi + 0.037\ ksi = 0.392 < 1.8\ ksi$ O.K.

As can be seen in the stress check in **Step 10**, the girder design for moment (which is the controlling parameter in sizing the girder) is not significantly affected by the 1" increase in deck thickness. For convenience in comparing, the total and LL+I stresses for the 8" and 9" composite decks are summarized in Table 6.14. As can be seen in that table, the total stresses in the steel girder are slightly increased while those in the deck are slightly decreased. The small increase in steel girder stresses are due to the small increase in the deck DL. It can be noted in Table 6.14 that the LL+I stresses are slightly reduced in both the steel girder and the concrete deck. This should translate into an increase in fatigue life of these components; and even thought the decrease is small (3 - 6%), because of the flatness of the S-N curves at small cyclic stress levels, this increase could be significant. Also it can be noted in Table B.14 that the short duration composite moment of inertia is increased from 21,142 in^4 to 22, 425 in^4 when going to the 9" deck. Thus the LL deflections for the 9" deck will be 0.94 of those of the 8" deck, i.e., there will be a 6% reduction in LL deflections.

Table B.14 Summary of Examples 3 and 3a Stresses for 8" and 9" Composite Decks

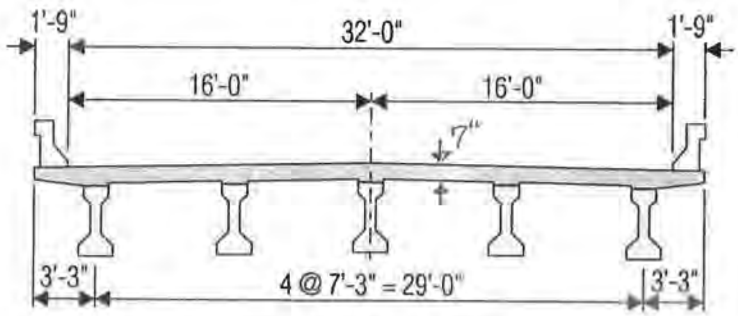
Loading	Parameter	8" Deck	9" Deck
	$I_{compos.}^{short\ term}$	21,142 in^4	22,425 in^{4*}
LL+I	$f_{bot.\ steel}$	9.34 ksi (T)	9.05 ksi (T)
	$f_{top\ steel}$	0.123 ksi (C)	0.127 ksi (T)**
	$f_{top\ conc.}$	0.373 ksi (C)	0.355 ksi (C)
Total	$f_{bot.\ steel}$	18.25 ksi (T)	18.65 ksi (T)
	$f_{top\ steel}$	7.84 ksi (C)	8.27 ksi (C)
	$f_{top\ conc.}$	0.412 ksi (C)	0.392 ksi (C)

$^*I_{8"} / I_{9"} = 21{,}142/22{,}425 = 0.94$

** Note that the total stress at this point does not go into tension due to the DL stress.

EXAMPLE B4 - Design of a Composite Prestressed Concrete Interior Girder

Problem Statement. Design the interior girder for the 70ft. simple span bridge shown below using a standard AASHTO PCI girder. Use working stress design (WSD). This example was extracted from Ref. ( ).



GIVEN:

- Simply supported span.
- Design span length = 70ft.
- Type III AASHTO-PCI girders.
- HS20-44 live loading.
- Barrier area = 2.61 ft^2.
- Steel: $f_s' = 270{,}000$ psi.
- Concrete: $f_c' = 5{,}000$ psi.
- 2.50 in wearing course.
- Deck & girder made of the same strength concrete.

STEP 1: Determine Impact and Distribution Factors

The impact factor is:

$$I = \frac{50}{L + 125} = \frac{50}{70\ ft + 125} = 0.26$$

➠Use I = 1.26

The distribution factor calculated for a bridge with:

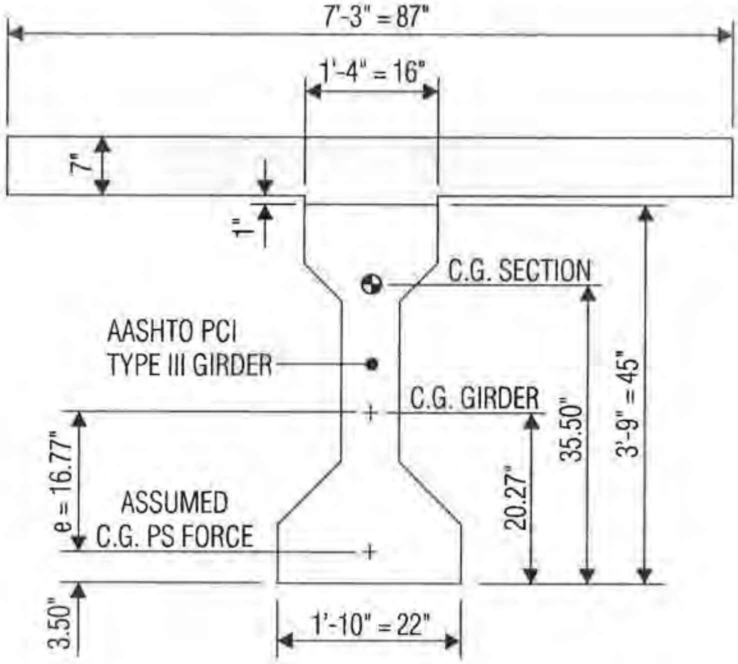
- Concrete Floor
- Two or More Traffic Lanes
- On Prestressed Concrete Girders

is,

$$DF = \frac{S}{5.5} = \frac{7.25\ ft.}{5.5} = 1.32$$

➠Use DF = 1.32

STEP 2: Calculate Moment of Inertia of Composite Section



ELEMENT	A (in^2)	Y (in)	AY (in^3)	AY^2 (in^4)	I_o (in^4)
SLAB	609.0	49.50	30145.5	1492202.250	2486.75
GIRDER	560.0	40.27	11351.2	230088.824	1257876.75
TOTALS	1169.0		41496.7	1722291.074	127876.75

$$I_Z = \Sigma I_O + \Sigma AY^2 = 127876.75 + 1722291.074 = 1850167.824\ in^4$$

$$Y' = \frac{\Sigma AY}{\Sigma A} = \frac{41496.7\ in^3}{1169\ in^2} = 35.498\ in$$

$$I = I_Z - (\Sigma A)(\Sigma Y')^2 = 1850167.8 - (1169)(35.5)^2 = 377{,}367\ in^2$$

STEP 3: Calculate Dead Load on Prestressed Girder
The dead load is composed of the following items:

$$DL_{slab} = (b_{eff})(slab\ thickness)(w_{conc})$$

$$= (7.25\ ft)(7\ in)(1\ ft/12\ in)(0.150\ k/ft^3) \qquad = 0.634\ k/ft$$

$$DL_{haunch} = (haunch\ width)(haunch\ thickness)(w_{conc})$$

$$= (1.33\ ft)(1\ in)(1\ ft/12\ in)(0.150\ k/ft) \qquad = 0.017\ k/ft$$

$$DL_{girder} = (girder\ area)(w_{conc})$$

$$= (560\ in^2)(1\ ft^2/144\ in^2)(0.150\ k/ft^3) \qquad = 0.583\ k/ft$$

$$DL_{barrier} = (2\ barriers)(barrier\ area)(w_{conc})\ /\ 5\ Girders$$

$$= (2)(2.61\ ft^2)(0.0150\ k/ft^3)\ /\ 5 \qquad = 0.157\ k/ft$$

$$DL_{wearing} = (w.c.\ thick.)(pave.\ width)(w_{pave})\ /\ 5\ Girders$$

$$= (2.5\ in)(1/12)(32\ ft)(0.150)\ k/ft^3)\ /\ 5 \qquad = 0.200\ k/ft$$

$$\Rightarrow DL = 1.591\ \text{k/ft}$$

STEP 4: Compute Dead Load Moments

Dead Load Moments:

$$M_{slab} = \frac{wL^2}{8} = \frac{(0.643+0.017)(70\ ft)^2}{8} \qquad \Rightarrow M_{slab} = 399\ ft\text{-}k$$

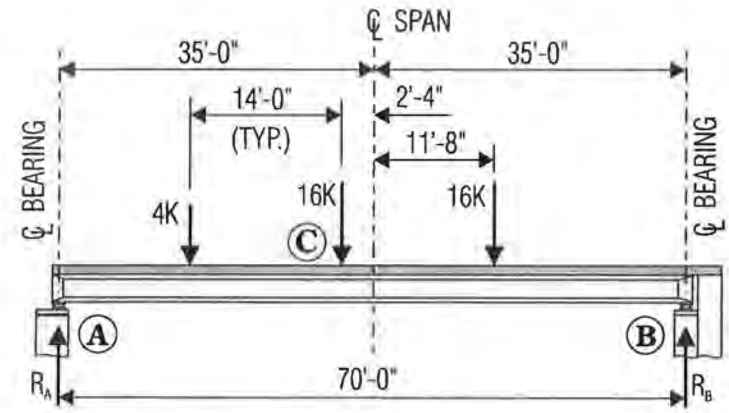
$$M_{girder} = \frac{wL^2}{8} = \frac{(0.538\ k/ft)(70\ ft)^2}{8} \qquad \Rightarrow M_{girder} = 357\ ft\text{-}k$$

$$M_{barrier} = \frac{wL^2}{8} = \frac{(0.157\ k/ft)(70\ ft)^2}{8} \qquad \Rightarrow M_{barrier} = 96\ ft\text{-}k$$

$$M_{wearing} = \frac{wL^2}{8} = \frac{(0.200\ k/ft)(70\ ft)^2}{8} \qquad \Rightarrow M_{wearing} = 123\ ft\text{-}k$$

$$\Rightarrow M_{DL} = 975\ \text{ft-k}$$

STEP 5: Calculate Live Load Plus Impact Moment
For Maximum moment the HS20-44 truck is located as shown below:



First, solve for the reactions by summing moments about Point A:

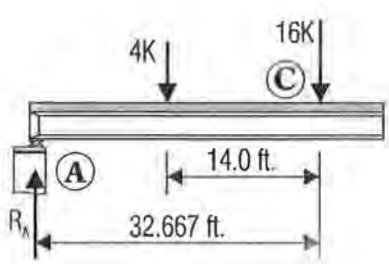
$\curvearrowright\!+\ \Sigma M_A + 0$:

$(4k \cdot 18.67\ ft) + (16\ k \cdot 3267\ ft) + (16\ k \cdot 46.67\ ft) - (R_B \cdot 70\ ft) = 0$

$$R_B = \frac{1344\ ft \cdot k}{70}\ ft = 19.20\ k \qquad so,\ R_A = 36\ k - 19.20\ k = 16.80\ k$$

Now, compute the maximum live load moment:

$M_{LL} = M_{MAX} = (R_A \cdot 32.67\ ft) - (4\ k \cdot 14\ ft) = 492.80\ ft \cdot k$



Apply the impact and wheel load distribution factors:

$M_{LL+I} = M_{LL} \cdot DF \cdot I$

$M_{LL+I} = (492.80\ ft\text{-}k)\ (1.32)\ (1.26)$

$\Rightarrow M_{LL+I} = 819.62\ ft\text{-}k$

STEP 6: Calculate Stresses at Top Fiber of Girder

Recapping from Step 2, the centroid distances and moments of inertia for the composite and noncomposite sections are:

NONCOMPOSITE Type III Girder	COMPOSITE 7" Slab & Type III Girder
$I = 125{,}390 \text{ in}^4$ $Y_t = (45\text{-}20.27) \text{ in} = 24.73 \text{ in}$ $Y_b = 20.27 \text{ in}$	$I = 377{,}367 \text{ in}^4$ $Y_t = (45\text{-}35.50) \text{ in} = 9.50$ $Y_b = 35.50 \text{ in}$

Stresses at the top fiber of the girder are calculated using the flexure equation (f = Mc / I):

Behavior	Element	Equation	Top Fiber
Noncomposite	Slab	$\dfrac{(399\ ft \cdot k)(12\ in\ /\ ft)(24.73\ in)}{125{,}390\ in^4}$	= 0.944 ksi
	Type III Girder	$\dfrac{(357\ ft \cdot k)(12\ in\ /\ ft)(24.73\ in)}{125{,}390\ in^4}$	= 0.845 ksi
Composite	LL+I	$\dfrac{(820\ ft \cdot k)(12\ in\ /\ ft)(9.50\ in)}{377{,}367\ in^4}$	= 0.248 ksi
	Barrier	$\dfrac{(96\ ft \cdot k)(12\ in\ /\ ft)(9.50\ in)}{377{,}367\ in^4}$	= 0.029 ksi
	Wear. Course	$\dfrac{(123\ ft \cdot k)(12\ in\ /\ ft)(9.50\ in)}{377{,}367\ in^4}$	= 0.037 ksi
		TOTAL f_{top}	= 2.103 ksi

STEP 7: Calculate Stresses at Bottom Fiber of Girder

Recapping from Step 2, the centroid distances and moments of inertia for the composite and noncomposite sections are:

NONCOMPOSITE Type III Girder	COMPOSITE 7" Slab & Type III Girder
$I = 125{,}390\ in^4$	$I = 377{,}367\ in^4$
$Y_t = (45\text{-}20.27)\ in = 24.73\ in$	$Y_t = (45\text{-}35.50)\ in = 9.50$
$Y_b = 20.27\ in$	$Y_b = 35.50\ in$

Stresses at the bottom fiber of the girder are calculated using the flexure equation (f = Mc / I)

Behavior	Element	Equation	Bottom Fiber
Noncomposite	Slab	$\frac{(399\ ft \cdot k)(12\ in\ /\ ft)(20.27\ in)}{125{,}390\ in^4}$	= 0.774 ksi
	Type III Girder	$\frac{(357\ ft \cdot k)(12\ in\ /\ ft)(20.27\ in)}{125{,}390\ in^4}$	= 0.692 ksi
Composite	LL+I	$\frac{(820\ ft \cdot k)(12\ in\ /\ ft)(35.50\ in)}{377{,}367\ in^4}$	= 0.926 ksi
	Barrier	$\frac{(96\ ft \cdot k)(12\ in\ /\ ft)(35.50\ in)}{377{,}367\ in^4}$	= 0.108 ksi
	Wear Course	$\frac{(123\ ft \cdot k)(12\ in\ /\ ft)(35.50\ in)}{377{,}367\ in^4}$	= 0.139 ksi
		TOTAL f_{bot}	= 2.639 ksi

STEP 8: Calculate Stress at Top of Concrete Deck

Composite 7" slab and Type III Girder

$I = 377{,}367\ in^4$

$Y_{td} = 9.5" + 1" + 7" = 17.5"$

Composite Action Longitudinal Stresses:

$$LL+I:\ \frac{820^k\ x\ 12^{"/'}\ x\ 17.5''}{377{,}367\ in^4} = 0.456\ ksi$$

$$Barrier:\ \frac{96^k\ x\ 12^{"/'}\ x\ 17.5''}{377{,}367\ in^4} = 0.053$$

$$Wear\ Course:\ \frac{123^k\ x\ 12^{"/'}\ x\ 17.5''}{377{,}367\ in^4} = 0.068\ ksi$$

Total $= 0.577$ ksi O.K.

STEP 9: Determine Girder Prestressing Forces, Stresses, Shears, Ultimate Flexural Capacity, etc.

Other parameters were checked and finalized. These are not shown here, as they are not significantly affected by a 1" increase in deck thickness.

This example design is repeated below with the deck thickness increased by 1", ie. from 7" to 8" to assess the effect of this deck design change on the support girders.

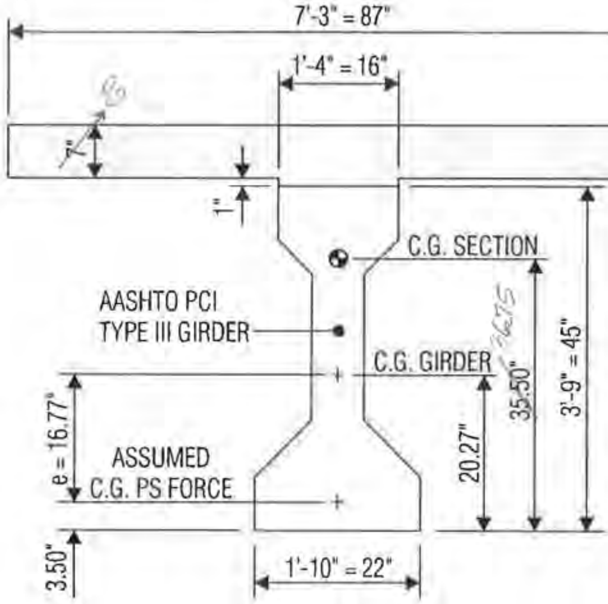
EXAMPLE B4a - Design Example B4 Modified for a 1" Thicker Deck

PROBLEM STATEMENT: Same problem statement as Example B4, but with a deck thickness of 8" rather than 7".

GIVEN: Same as Example B4 except that the deck thickness is 8", rather than 7".

STEP 1: Determine Impact and Distribution Factors
(No change)

STEP 2: Calculate Moment of Inertia of Composite Section



ELEMENT	A (in^2)	Y (in)	AY (in^3)	AY^2 (in^4)	I_o (in^4)
SLAB	696.0	50.00	34,800.0	1,740,000.0	3712.0
GIRDER	560.0	20.27	11351.2	230,088.824	125,390.00
TOTALS	1256.0		46,151.2	1,970,088.82	129,102.0

$$I_Z = \Sigma I_O + \Sigma AY^2 = 129{,}102 + 1{,}970{,}088.82 = 2{,}099{,}190.82\ in^4$$

$$Y' = \frac{\Sigma AY}{\Sigma A} = \frac{46{,}151.2\ in^3}{1256\ in^2} = 36.75\ in$$

$I = I_z - (\Sigma A)(\Sigma Y')^2 = 2{,}099{,}190.82 - (1256)(36.75)^2 = 403{,}365 \text{ in}^4$

STEP 3: Calculate Dead Load on Prestressed Girder

The dead load is composed of the following items:

$DL_{slab} = (b_{eff})\text{ (slab thickness) }(w_{conc})$

$= (7.25 \text{ ft})(\overset{8}{\cancel{7}} \text{ in})(1 \text{ ft}/12 \text{ in})(0.150 \text{ k/ft}^3) = \cancel{0.634 \text{ k/ft}} \rightarrow 0.725 \text{ k/ft}$

$DL_{haunch} = \text{(haunch width) (haunch thickness) }(w_{conc})$

$= (1.33 \text{ ft})(1 \text{ in})(1 \text{ ft}/12 \text{ in})(0.150 \text{ k/ft}^3) = 0.017 \text{ k/ft}$

$DL_{girder} = \text{(girder area) }(w_{conc})$

$= (560 \text{ in}^2)(1 \text{ ft}^2/144 \text{ in}^2)(0.150 \text{ k/ft}^3) = 0.583 \text{ k/ft}$

$Dl_{barrier} = \text{(2 barriers) (barrier area) }(w_{conc}) / 5 \text{ Girders}$

$= (2)(2.61 \text{ ft}^2)(0.150 \text{ k/ft}^3)/5 = 0.157 \text{ k/ft}$

$Dl_{wearing} = \text{(w.c. thick.)(pave. width)}(w_{pave}) / 5 \text{ Girders}$

$= (2.5 \text{ in})(1/12)(32 \text{ ft})(0.150 \text{ k/ft}^3)/5 = \underline{0.200 \text{ k/ft}}$

$\Rightarrow DL = \cancel{1.591 \text{ k/ft}} \rightarrow 1.682 \text{ k/ft}$

STEP 4: Compute Dead Load Moments

Dead Load Moments:

$$M_{slab} = \frac{wL^2}{8} = \frac{(\overset{0.725}{\cancel{0.634}} + 0.017)(70\ ft)^2}{8} \quad \Rightarrow M_{slab} = \cancel{399\ ft\text{-}k} \rightarrow 454\ ft\text{-}k$$

$$M_{girder} = \frac{wL^2}{8} = \frac{(0.583\ k/ft)(70\ ft)^2}{8} \quad \Rightarrow M_{girder} = 357\ ft\text{-}k$$

$$M_{barrier} = \frac{wL^2}{8} = \frac{(0.157\ k/ft)(70\ ft)^2}{8} \quad \Rightarrow M_{barrier} = 96\ ft\text{-}k$$

$$M_{wearing} = \frac{wL^2}{8} = \frac{(0.200\ k/ft)(70)\ ft)^2}{8} \quad \Rightarrow M_{wearing} = 123\ ft\text{-}k$$

$\Rightarrow M_{DL} = \cancel{975 \text{ ft-k}} \rightarrow 1030 \text{ ft-k}$

STEP 5: Calculate Live Load Plus Impact Moment

(No Change)

STEP 6: Calculate Stresses at Top Fiber of Girder

Recapping from Step 2, the centroid distances and moments of inertia for the composite and noncomposite sections are:

NONCOMPOSITE Type III Girder	COMPOSITE 7" Slab & Type III Girder
$I = 125{,}390\ in^4$	$I = 403{,}365\ in^4$
$Y_t = (45\text{-}20.27)\ in = 24.73\ in$	$Y_t = (45\text{-}36.75)\ in = 8.25$
$Y_b = 20.27\ in$	$Y_b = 36.75\ in$

Stresses at the top fiber of the girder are calculated using the flexure equation (f = Mc/I):

Behavior	Element	Equation	Top Fiber
Noncomposite	Slab	$\dfrac{(454\ ft \cdot k)(12\ in\ /\ ft)(24.73\ in)}{125{,}390\ in^4}$	= 1.074 ksi
	Type III Girder	$\dfrac{(357\ ft \cdot k)(12\ in\ /\ ft)(24.73\ in)}{125{,}390\ in^4}$	= 0.845 ksi
Composite	LL+I	$\dfrac{(820\ ft \cdot k)(12\ in\ /\ ft)(8.25\ in)}{403{,}365\ in^4}$	= 0.201 ksi
	Barrier	$\dfrac{(96\ ft \cdot k)(12\ in\ /\ ft)8.25\ in)}{403{,}365\ in^4}$	= 0.024 ksi
	Wear. Course	$\dfrac{(123\ ft \cdot k)(12\ in\ /\ ft)(8.25\ in)}{403{,}365\ in^4}$	= 0.030 ksi
		TOTAL f_{top}	= 2.174 ksi

STEP 7: Calculate Stresses at Bottom Fiber of Girder

Recapping from Step 2, the centroid distances and moments of inertia for the composite and noncomposite sections are:

NONCOMPOSITE Type III Girder	COMPOSITE 7" Slab & Type III Girder
$I = 125{,}390\ in^4$ $Y_t = (45\text{-}20.27)$ in = 24.73 in $Y_b = 20.27$ in	$I = 403{,}365\ in^4$ $Y_t = (45\text{-}36.75)$ in = 8.25 $Y_b = 36.75$ in

Stresses at the bottom fiber of the girder are calculated using the flexure equation (f = Mc/I):

Behavior	Element	Equation	Bottom Fiber
Noncomposite	Slab	$\dfrac{(454\ ft \cdot k)(12\ in\ /\ ft)(20.27\ in)}{125{,}390\ in^4}$	= 0.881 ksi
	Type III Girder	$\dfrac{(357\ ft \cdot k)(12\ in\ /\ ft)(20.27\ in)}{125{,}390\ in^4}$	= 0.692 ksi
Composite	LL+I	$\dfrac{(820\ ft \cdot k)(12\ in\ /\ ft)(36.75\ in)}{403{,}365\ in^4}$	= 0.897 ksi
	Barrier	$\dfrac{(96\ ft \cdot k)(12\ in\ /\ ft)(36.75\ in)}{403{,}365\ in^4}$	= 0.105 ksi
	Wear Course	$\dfrac{(123\ ft \cdot k)(12\ in\ /\ ft)(36.75\ in)}{403{,}365\ in^4}$	= 0.134 ksi
		TOTAL f_{top}	= 2.709 ksi

STEP 8: Calculate stress at top of concrete deck:
(in longitudinal direction)

Composite 8" slab and Type II Girder:

$I = 403{,}365\ in^4$

$Y_{td} = 8.25" + 1" + 8" = 17.25"$

Composite Action Stresses:

$$LL+I: \quad \frac{820'^k \; x \; 12''/' \; x \; 17.25''}{403{,}365 \; in^4} = 0.421 \; ksi$$

$$Barrier: \quad \frac{96'^k \; x \; 12''/' \; x \; 17.25''}{403{,}365 \; in^4} = 0.049 ksi$$

$$Wear \; Course: \quad \frac{123'^k \; x \; 12''/' \; x \; 17.25''}{403{,}365 \; in^4} = 0.063 \; ksi$$

Total = 0.533 ksi O.K.

Note, in the transverse direction, the concrete stresses for the 8" slab will be approximately,

$$\frac{(7^2)}{(8^2)} = \frac{49}{64} = 0.765$$

of those of the 7" slab. This is significant reduction, and will result in less longitudinal cracking, smaller crack widths, and greater fatigue life.

STEP 9: Determine Girder Prestressing Forces, Stresses, Shears, Ultimate Flexural Capacity, etc.

(No Significant Change)

As in the composite steel girder design of Example B.3, this example employing a composite prestressed girder design is not significantly affected by the 1" increase in deck thickness. This can be seen in the stress summaries shown for comparative convenience in Table B.15. The same Type III AASHTO girders would be used for either the 7" or 8" thick deck. The same observations and comments made at the end of Example B4 are applicable for Example B4a.

Table B.15 Summary of Examples B4 and B4a Stresses for 7" and 8" Composite Decks

Loading	Parameter	7" Deck	8" Deck
-	$I_{compos.}^{short\ term}$	377,367 in^4	403,365 in^4*
Deck + Girder DL (Noncomposite)	$f_{top\ of\ girder}$	1.789 ksi (C)	1.919 ksi (C)
	$f_{bot\ of\ girder}$	1.466 ksi (T)	1.573 ksi (T)
LL+I	$f_{top\ of\ girder}$	0.248 ksi (C)	0.201 ksi (C)
	$f_{bot\ of\ girder}$	0.926 ksi (T)	0.897 ksi (T)
	$f_{top\ of\ deck}$	0.456 ksi (C)	0.421 ksi (C)
Total	$f_{top\ of\ girder}$	2.103 ksi (C)	2.174 ksi (C)
	$f_{bot\ of\ girder}$	2.639 ksi (T)	2.709 ksi (T)
	$f_{top\ of\ deck}$	0.577 ksi (C)	0.533 ksi (C)

*$I_{7"}/I_{8"}$ = 377,367 / 403,365 = 0.94

EXAMPLE B5 - Noncomposite Rolled Steel Girder Bridge

Problem Statement. Design the simple span noncomposite rolled steel girder bridge of Figure B.8 with 30.5-mm span for a *HL*-93 live load. Roadway width is 13 420-mm curb to curb. Allow for a future wearing surface of 75-mm thick bituminous overlay. Use f'_c =30 MPa and M270 Gr345 steel. Design in compliance with AASHTO (1994) LRFD Bridge Specifications.

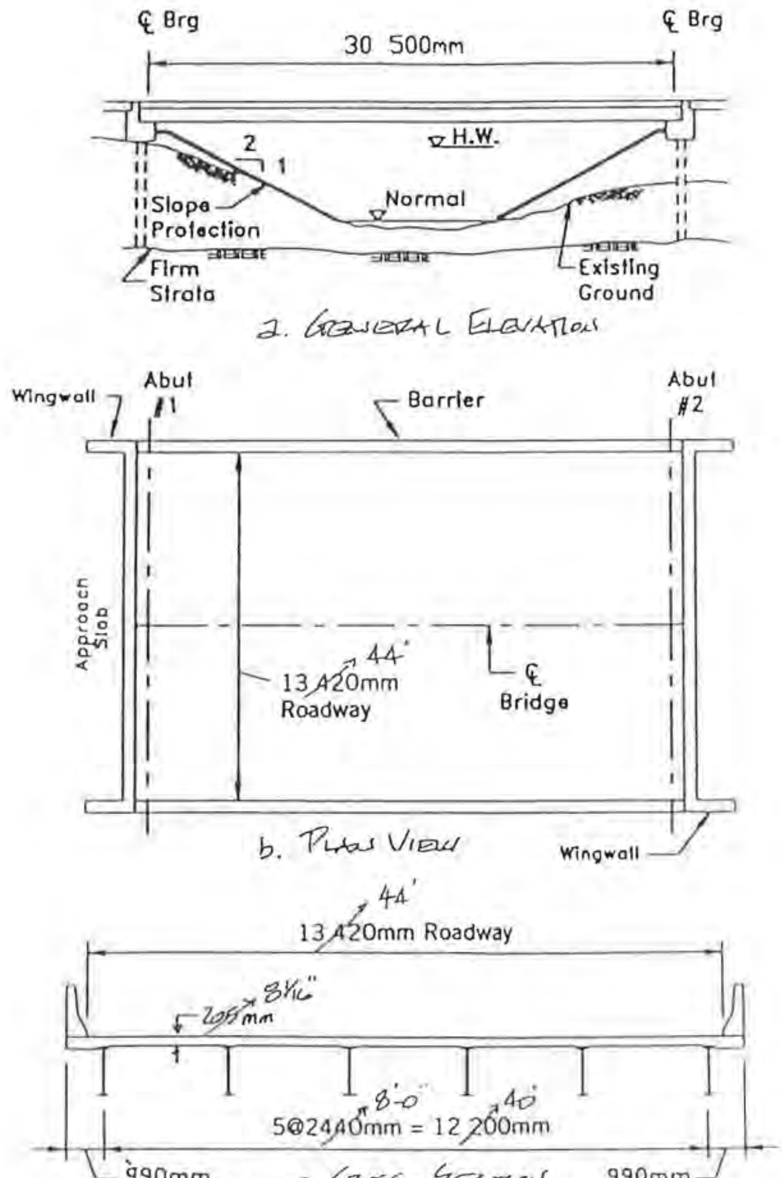


Fig. B.8 Noncomposite rolled steel girder bridge design example ().

For a uniformly distributed load, w,

$$V_{END} = V_{MAX} = \frac{wL}{2} = \frac{w(30.5)}{2} = 15.25w$$

$$M_{cen} = M_{max} = \frac{wL^2}{8} = \frac{w(30.5)^2}{8} = 116.28w$$

DEAD LOAD ANALYSIS:

Assume a girder weight of 5 kN/m.

1. *Interior Girders*

DC	Deck Slab	$(2400)(10^{-9})(9.81)(205)(2440) = 11.78$ kN/m
	Girder	= 5.0 kN/m
		$w_{DC} = 16.78$ kN/m

DW 75-mm bituminous paving = $(2250)(9.81)(10^{-9})(75)(2440)$

$$w_{DW} = 4.04 \text{ kN/m}$$

2. *Exterior Girders.* Using deck design results for reaction on exterior girder,

DC	Deck Slab	4.60 kN/m
	Overhang	6.75 kN/m
	Barrier	6.74 kN/m
	Girder	5.0 kN/m
		$w_{DC} = 23.09$ kN/m

DW 75-mm bituminous paving

$$w_{DC} = (2250)(10^{-9})(9.81)(75)(610+\frac{2440}{2})$$

$$=3.03\text{kN/m}$$

LIVE LOAD MOMENT ANALYSIS:

See Fig. B.9.

$$M_{LL+IM} = \underset{\substack{\uparrow \\ \text{Distribution} \\ \text{Factor}}}{mg}\left[\left(M_{Truck}\ or\ M_{Tandom}\right)\left(1+\frac{IM}{100}\right)+M_{Lane}\right]$$

$$M_{Truck} = 170.25(11.678) + (25.25)(4.3) = 2077\ kN\ m \quad \leftarrow \text{governs}$$

$$M_{Tandom} = 110(14.65) = 1611.5\ kN\ m$$

$$M_{Lane} = \frac{9.3(30.5)^2}{8} = 1081.41\ kN\ m$$

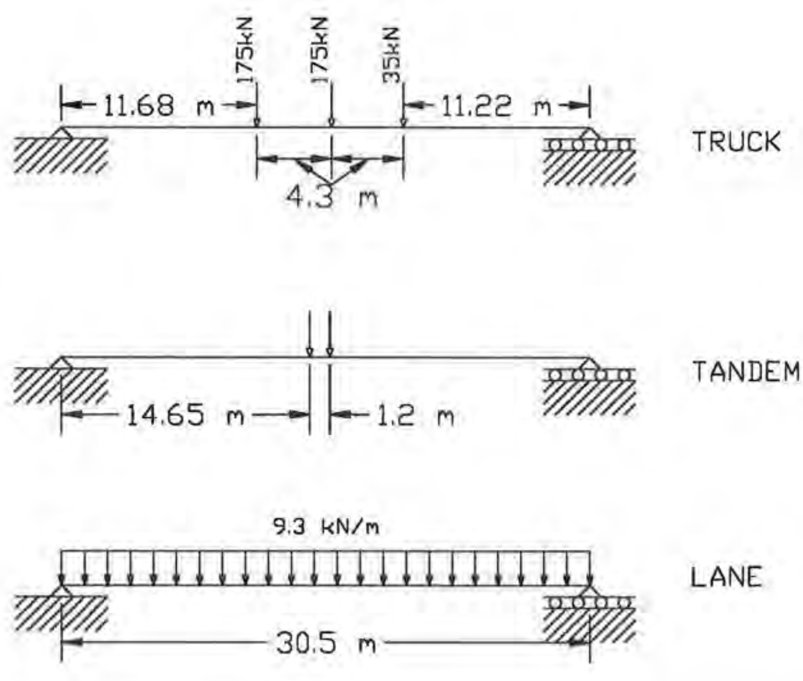


Fig. B.9 Truck, tandem, and lane load placement for maximum moment at midspan.

Therefore,

Interior Girders

$$M_{LL+IM} = 0.748[2007(1.33) + 1081.41] = 2875\ kN\ m$$

Exterior Girders

$$M_{LL+IM} = 0.762[2077(1.33) + 1081.41] = 2929\ kN\ m$$

Shear does not control (which is typical) and is not shown.

Unfactored maximum moments and shears for interior and exterior girders are shown in Table B.16 below.

TABLE B.16 Unfactored Maximum Moments and Shears

Girder	Load Type	*W(kN/m)*	Moment (kN m) M_{105}	Shear (kN) V_{100}
Interior	*DC*	16.78	1952	256
	DW	4.04	469	62
	LL + IM	N/A	2875	401
Exterior	*DC*	23.09	2685	352
	DW	3.03	352	46
	LL + IM	N/A	2929	407

GIRDER SECTION REQUIRED:

Strength I limit State

a. Interior girder

Factored shear and moment

$U = \eta[1.25DC + 1.50DW + 1.75(LL + IM)]$

$V_u = 0.95[1.25(256) + 1.50(62) + 1.75(401)]$

$= 1059$ kN

$M_u = 0.95[1.25(1952) + 1.50(469) + 1.75(2875)]$

$= 7766$ kN m

b. Exterior girder

Factored shear and moment

$V_u = 0.95[1.25(352) + 1.50(46) + 1.75(407)]$

$= 1160$ kN

$M_u = 0.95[1.25(2685) + 1.50(352) + 1.75(2929)]$

$= 8560$ kN m $\leftarrow$ controls

Note that the exterior girder controls the girder size/design for this example.

Required Plastic Section Modulus, Z

$$\phi_f M_n \geq M_u,\ \ \phi_f = 1.0,\ M_n = M_p = Z\,F_y$$

$$Z\,F_y \geq M_u$$

Assuming compression flange is fully braced and section is compact,

$$r'eqd\ Z \geq \frac{M_u}{F_y} = \frac{8560\,x\,10^6}{345} = 24.812\,x\,10^6\ mm^3$$

Try W1100 x 499 , Z = 26.60 x 10^6 mm^3, S = 23100 x 10^3 mm^3, I = 12900 x 10^6 mm^4,

w_g = (499)(9.81)(10^{-3})

= 4.9 kN/m ← Less than the assumed value of 5 but close. Therefore, O.K.

Note, W1100 x 499 → W44 x 335

Shear capacity checks out for this girder section, as do service load requirements.

EXAMPLE 5a - Design Example 1 Modified for a 1" Thicker Deck

Problem Statement. Let us now examine the effect of increasing the deck thickness 1" on the girder design for the bridge of Example B5. Example B5 is repeated below with a 1" thicker deck to make this assessment.

Assume the deck thickness is increased from 205 mm to 230mm

Therefore,

INTERIOR

w_{DC} = 13.21 Deck

5 Girder (Assumed)

18.21 kN/m

EXTERIOR

w_{DC} = 5.16 Deck

7.57 Overhang

6.74 Barrier

5.0 Girder (Assumed)

24.47 kN/m

Therefore, the dead loads, moments, and shears in the Table B.16 would be altered as indicated in Table B.17.

TABLE B.17 Unfactored Maximum Moments and Shears for Thickened (230mm) Deck

Girder	Load Type	*W(kN/m)*	Moment (kN m) M_{105}	Shear (kN) V_{100}
Interior		18.21	2118	278
	DC	16.78	1952	256
	DW	4.04	469	62
	LL + IM	N/A	2875	401
		24.47	2845	373
Exterior	*DC*	23.09	2685	352
	DW	3.03	352	46
	LL + IM	N/A	2929	407

Therefore, the new Strength I Limit State design values for the girders are as follows.

Interior Girder:

$$V_u = 0.95\,[1.25\,(\overset{278}{256}) + 1.50\,(62) + 1.75\,(401)]$$

$$= 1085\text{kN}$$

$$M_u = 0.95\,[1.25\,(\overset{2118}{1952}) + 1.50\,(459) + 1.75\,(2875)]$$

$$= 7963\text{kNm}$$

Exterior Girder:

$$V_u = 0.95\,[1.25\,(\overset{373}{352}) + 1.50\,(46) + 1.75\,(407)]$$

$$= 1185\text{ kN}$$

$$M_u = 0.95\,[1.25\,(\overset{2845}{2685}) + 1.50\,(352) + 1.75\,(2929)]$$

$$= 8750\text{ kNm} \qquad \leftarrow \text{controls}$$

$$Z_{REQD} \geq \frac{M_u}{F_y} = \frac{8750x10^6}{345} = 25.36x10^6\ mm^3$$

Again,

W1100x499 ($Z = 26.60\text{x}10^6\ mm^3$)

Thus, use W1100 x 499 or W44 x 335 as before.

Note that increasing deck thickness by 1", caused the controlling girder ultimate moment (factored moment) to increase from

$$M_u = 8560\text{kNm} \quad \xrightarrow{\text{to}} \quad 8750\text{ kNm}$$

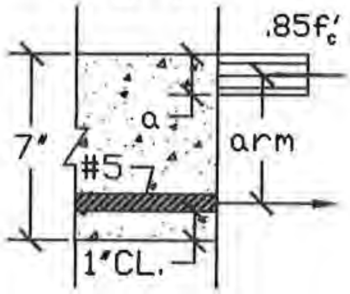
This is only a 2.22 increase in design moment, and as can be seen above, does not affect the final girder sizing/design.

Appendix C

Example Illustrating Additional Cost of Deck Concrete can be Captured in Reduced Deck Rebar Cost

From ALDOT's Structures Design and Detail manual,

For S = 8' $\big\}$ *Transverse Steel* $\Rightarrow$ #5 @ 5" *o.c.* $\Rightarrow A_5 = 0.74\ in^2/ft$ *(top & bottom)*
D = 7"



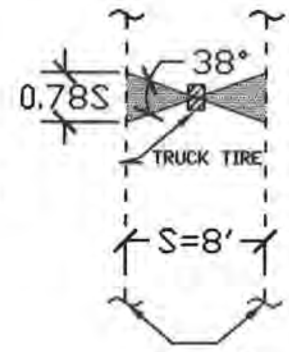
$$C = 0.85 f_c' a \overset{12"}{b}$$

$$T = A_s f_y = 0.74 in^2\ x\ 60 k/in^2 = 44.4^k$$

$$C = 0.85 \overset{4\ ksi}{f'_c} a\ (12") = T = 44.4\, k$$

$$a = 1.088"$$

$$arm = 7.0" - 1.0" - 0.3125" - 0.544" = 5.14"$$



GIRDER

PLAN VIEW

If increase deck thickness to 8" $\rightarrow W_{DL\ Deck}$ increases slightly but assume the resulting increasein M_{DL} is negligible relative to M_{LL+I} as indicated below.

$$P_{Truck\ Tire} = \begin{array}{r} 16^k \\ \underline{x\ 1.3}\ \text{IF.} \\ 20.8^k \end{array} \quad \rightarrow \quad M_{LL+I} = \frac{PL}{4} = \frac{20.8^k\ x\ 8'}{4} = 41.6^{1k}$$

$$\frac{M_{LL+I}}{ft} = \frac{41.6^{1k}}{0.78\ x\ 8'} = 6.7\ 'k/ft$$

$$M_{DL} = \frac{WL^2}{8} = \frac{(\frac{7}{12}\ x\ 1\ x\ 150)8^2}{8} = \underline{0.7'k/ft}$$

$$7.4\ 'k/ft$$

$$\Delta\ M_{DL} = \frac{(\frac{1}{12}\ x\ 1\ x\ 150)\ 8^2}{8} = \underline{0.1'k/ft}$$

$$7.4\ 'k/ft$$

$$\frac{0.1}{7.4}\ x\ 100 = 1.4\%$$

Increase of M_{trans} which is negligible. In new construction with SIP metal forms, the forms carry the deck weight, and thus M_{DL} and ΔM_{DL} are both zero.

Therefore,

$$\text{arm}^{\text{for 8"DECK}} = \text{arm}^{\text{7"DECK}} + 1.0" = 6.14"$$

Recall

$$\left.\begin{aligned} A_s^{7"} &= \frac{M_{factored\ design}}{f_y\ x\ 5.14"} \\ A_s^{8"} &= \frac{M_{factored\ design}}{f_y\ x\ 6.14"} \end{aligned}\right\}$$

$M_{\text{factored design}}$ Approx. the same, therefore,

$$\frac{A_s^{8"}}{A_s^{7"}} = \frac{5.14"}{6.14"} = 0.837$$

$$A_s^{8"} = 0.847\ x\ A_s^{7"}$$

$$A_s^{8"} = 0.837\ x\ 0.74 = 0.62\ in^2/ft$$

Therefore,

$\Delta A_s^{7\to 8} = -0.12\ in^2/ft$ for both top & bottom transverse bars

$\Delta A_s^{Total\ Trans} = 2\ \Delta A_s = -0.24\ in^2/ft$ in transverse direction

Note in longitudinal direction will use same distribution and temperature steel, therefore,

$$\Delta A_s^{Longit.} = 0$$

$$\Delta A_s^{Total} = \Delta A_s^{Tran.} + \Delta A_s^{Longit.} = -0.24 + 0 = -0.24\ in^2/ft$$

For 38' wide x 100' long bridge,

$$\Delta W_T^{Steel} = -\frac{0.24\ in^2/ft}{144}\ x\ 38'\ x\ 100'\ x\ \frac{490^{\#}}{ft^3} = -3103^{\#}$$

$$\Delta Cost^{Steel} = -3103^{\#}\ x\ \frac{\$0.50}{\#} = -\$1552$$

$$\Delta W_T^{Concrete} = \underbrace{(\frac{1}{12})\ x\ 38\ x\ 100}_{317\ ft^3}\ x\ \frac{150^{\#}}{ft^3} = 47{,}500^{\#}$$

$$\Delta Cost^{Concrete} = \frac{317ft^3}{27\frac{ft^3}{yd^3}}\ x\ \frac{\$70}{yd^3} = \$821$$

Therefore,

$\Delta \text{Cost}^{\text{Total}} = -\$1552 + \$821 = -\$731$ or a <u>$731 Savings</u>