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PERFORMANCE OF CONCRETE BRIDGE DECK SLABS USING ISOTROPIC AND STANDARD AASHTO PROCEDURES

University-Based Research, Education, and Technology Transfer Program
AGREEMENT NO. 359704, WORK ORDER 24

FINAL REPORT

October 2001

By G. Sabnis

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16. Abstract

This report describes the work done with alternate means of designing deck slabs by reducing the amount of reinforcement, without sacrificing the integrity of serviceability and strength of the decks, could result in considerable cost reduction and efficient solution. The fundamental concept in the isotropic deck design is based on the "arching" effect, which takes place in the concrete deck between the short spans of the beams. As a result, the steel reinforcement experiences insignificant flexural stress. The arching effect causes the deck slab to fail in punching shear rather then flexural failure, which is the basis of the design of conventional AASHTO deck slabs. Therefore, shear and temperature governs the amount of reinforcement needed. Pennsylvania Department of Transportation (PENNDOT) conducted field demonstration projects in which bridge decks were designed. They were evaluated over a five-year period. Each isotropic deck constructed had a matching counterpart (control) designed with AASHTO standards and were inspected along with those designed using empirical procedures after the first, third, and fifth year of construction. A total of seven isotropic and six conventional AASHTO decks were constructed. Evaluation of the performance of the decks is based on field monitoring reports, which give details of crack occurrence, spalling of the decks, and general condition of the bridge deck slab with actual field visit to various sites and comparing field behavior. Based on these field studies, as well as on analytical studies conducted during this research, isotropic decks performed satisfactory in accordance with the AASHTO standards. The arrangement of isotropic reinforcement is much simpler than the reinforcement pattern of AASHTO decks, resulting in easier and faster construction. Reduction in steel reinforcement is decks using isotropic design makes an attractive proposal for the state DOTs

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Prepared for

Commonwealth of Pennsylvania
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ABSTRACT

Bridge decks are generally designed using the accepted practices of the conventional American Association of State Highway and Transportation Officials (AASHTO) code. The current AASHTO design code, however, usually overestimates the stresses in the steel reinforcement of bridge decks; therefore, an alternate means of reducing the amount of reinforcement without sacrificing the integrity of serviceability and strength of the decks could result in considerable cost reduction and efficient solution. This reduction in steel reinforcement may be achieved by the use of isotropic decks that are designed using the empirical procedure. The fundamental concept in the isotropic deck design is based on the 'arching' effect that takes place in the concrete deck between the short spans of the beams. As a result, the steel reinforcement experiences insignificant flexural stress. The arching effect causes the deck slab to fail in punching shear rather than flexural failure, which is the basis of the design of conventional AASHTO deck slabs, therefore, shear and temperature governs the amount of reinforcement needed.

To use this concept successfully in the construction of bridge decks, Pennsylvania Department of Transportation (PENNDOT) conducted an experimental demonstration project in which bridge decks were designed, and then they were evaluated over a five-year period. Each isotropic deck constructed had a matching counterpart(control) designed with AASHTO standards and were inspected along with those designed using empirical procedures after the first, third, and fifth years of construction. A total of seven isotropic and six conventional AASHTO decks were constructed. Evaluation of the performance of the decks is based on field monitoring reports, which give details of crack

occurrence, spalling of the decks, and general condition of the bridge deck slab with actual field visit to various sites and comparing field behavior.

Based on these field studies, as well as on analytical studies conducted during this research, isotropic decks performed satisfactorily in accordance with the AASHTO standards. The arrangement of isotropic reinforcement is much simpler than the reinforcement pattern of AASHTO decks, resulting in easier and faster construction. Reduction in the amount of steel reinforcement over conventional AASHTO decks also makes isotropic deck cheaper to build.

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CHAPTER 1. INTRODUCTION

1.1 Problem Definition

Experience has shown that the current American Association of State Highway and Transportation Officials (AASHTO) procedures for the design of bridge decks overestimate the stresses in the steel reinforcement. The current design methods are based on flexural analysis in the decks; however, it has been shown through a number of experiments that the decks designed by the current AASHTO method fail in punching shear rather than in flexure, and the failure load is normally much higher than predicted by calculations. Alternatively, the design concept for reinforcing isotropic bridge decks is the 'arching effect,' which takes place in the short span of the deck between the beams. Because of this action, the reinforcing steel experiences negligible flexural stresses; therefore, shear and temperature stresses govern the design of the reinforcement in the deck slab. The arching action, also referred to as the compressive membrane action, is especially evident in decks with lateral restraints. Such lateral restraints are enhanced by the addition of shear studs when steel beams are used, but studs are usually not necessary when concrete beams are utilized. The studs help the slab to act compositely with the beams. Because the stresses in the reinforcement are significantly overestimated when the decks are laterally restrained, it may be beneficial to make use of the isotropic deck slab in order to optimize the use of the steel reinforcement while maintaining the structural integrity of the decks. To verify the effectiveness of isotropic decks compared to AASHTO decks under real conditions, Pennsylvania Department of Transportation (PENNDOT) constructed seven isotropic decks with all but one having an AASHTO counterpart.

The field investigation was carried over a five-year period. The decks were inspected in the first, third, and fifth years for cracking and spalling and any other visible defects. Comparison was then made between the performance of the two types of decks. Another critical judging criterion was cost, since the primary objective was the reduction in cost of the decks due to reduction in steel, which is expected to further decrease labor cost.

1.2 The Fundamental Concept in Isotropic Deck Design

An isotropic deck slab is one in which the flexural rigidity in the primary orthogonal directions and half the torsional rigidity are equal (Cusens and Pama 1975). The design utilizes the theory that an 'arching effect' takes place in the short span of the concrete deck slab, as indicated in section 1.1. This phenomenon arises when the slab is laterally restrained by the external forces, such as the resistance due to flexurally stiff external girders. This can be visualized as the slab jamming against the external beams when the slab tends to bend (Park and Gamble 2000). The external beams thus impose a compressive force on the slab. The arching effect in concrete is also apparent in column/slab buildings (Ouyan and Suaris1987). Tests carried out on buildings that were being demolished revealed that the slab strength was much greater than what was predicted by the flexural design method. This phenomenon was explained by the compressive membrane action.

1.3 Research Objectives

The objective of this research is to compare the analysis and behavior of bridge decks using empirical and standard procedures based on the current AASHTO code. The behavior at the serviceability stage of the two types of decks is compared using actual field data in various Districts of Pennsylvania over a period of 1 to 10 years. Another important aspect of this research is to make recommendations to the Pennsylvania Department of Transportation on the use of isotropic decks.

1.4 Research Significance

Bridges account for a significant fraction of the cost of most road projects; hence, any means of reducing the cost significantly without compromising durability, safety, and strength would be worth investigating. Not only is there a need for new bridges as the nation's population expands quickly and more land has to be inhabited, but many of the present bridge decks are decrepit, and, consequently, need to be replaced. Presently, there are about 25,000 bridges in the state of Pennsylvania, the majority of which are AASHTO-type decks. Replacement of these decks with isotropic decks may result in a saving of several million dollars over the present AASHTO method. Not only will the use of isotropic decks possibly result in cheaper bridge decks, but it may help in the optimization of steel reinforcement in concrete bridge deck slabs.

1.5 Scope

Pennsylvania Department of Transportation (PENNDOT) constructed a total of 13 bridges for this study from 1990 to 1993. Seven of the constructed bridges were isotropic decks while the other six were AASHTO types. A list of those decks is shown in section 2.2 of chapter 2. The bridges were constructed so that each isotropic deck experienced similar traffic and environmental conditions as its AASHTO counterpart. The counterparts for each deck did not necessarily have the same span but were constructed so that their performance would be similar. The performance of the decks is based on their strength and serviceability. The strength is based on cracking moment and shear capacity, while serviceability is based on spalling and crack occurrence on the decks, but there is no established standard defining level of performance based on the aforementioned criteria.

The decks were investigated over a period of five years, with investigations in the first, third, and fifth years after construction; however, follow-up field investigations performed after that period were included in the study. Each deck was inspected for cracking and spalling, and any other critical defects. During the field inspection, sketches were made of the decks, and crack patterns and decks were photographed. A general condition report of each deck was written following the field work. The type and location of the cracks were also noted. They were classified as transverse cracks, longitudinal cracks, and diagonal cracks. In addition to crack pattern, any permit load on the decks was noted.

The work done by PENNDOT field inspectors is analyzed and summarized in this thesis. Suitable recommendations will be made concerning the utilization of isotropic decks by the state after extensive review of the PENNDOT literature and available research materials. The cause of deterioration in bridge decks will be reviewed along with the relevant laboratory test data. The AASHTO and isotropic deck design procedure will also be compared.

Although an ideal isotropic deck has the same primary reinforcement in both principal orthogonal directions, the isotropic decks in this investigation do not necessarily have the same reinforcement in both principal directions. The major attribute consistent with these 'isotropic decks' is the low reinforcement ratio compared to their AASTHO counterparts. Therefore, a more appropriate term for these decks is 'lightly reinforced decks; however, where the term isotropic bridge deck is used in this document, it generally means lightly reinforced concrete deck slab.

CHAPTER 2. REVIEW OF LITERATURE AND AVAILABLE DOCUMENTS FROM PENNDOT

This chapter is presented in two main sections. The first section explores factors affecting the performance of bridge decks and further exploration of the arching effect and its applications to laboratory and prototypes concrete bridge decks. The latter section is a review and summary of the available PENNDOT documents.

2.1 Background

Isotropic bridge decks have been utilized in Ontario, Canada, prior to their experimentation in the United States. The benefits of the arching effect have been utilized in the Ontario Highway Bridge Design Code as far back as 1979. In fact, early in the investigation of such decks in the United States, they were called "Ontario-type decks". Currently, there are isotropic decks in Texas, New York, Michigan, Oregon, Wyoming, and Wisconsin in addition to those in Pennsylvania, all of them constructed on an experimental basis. Isotropic decks consist of two mats of steel, with the top steel mat identical to the bottom steel mat and are aligned in the two principal orthogonal directions of the decks. Consequently, the flexural rigidity in two principal orthogonal directions of the decks is identical; however, where there is a certain degree of skewness, the reinforcements are modified. This design concept can reduce the amount of reinforcement by as much as 40% to 60 % compared to that using the AASHTO design for standard loading.

2.1.1 Cracking of Concrete Bridge Decks

The deterioration of bridge decks usually starts with the onset of cracking. Cracks are formed in concrete when the tensile stresses are greater than the tensile strength of the concrete. These stresses are induced mainly by restraint of volume changes and external loads, and the level of stress is directly proportional to the magnitude of the volume change and inversely proportional to the extensibility of the concrete during the plastic stage. The ductility of the concrete is only being referred to during the plastic stage; when concrete is cured, it becomes a brittle material. Consequently, its extensibility is zero for all practical purposes. In the plastic stage, volume change may occur when the rate of evaporation of the water in the surface of the concrete is greater than the rate at which bleed water from the bottom layer replaces it. As a result, the concrete tends to shrink, but the layer of concrete in the lower region prevents it from doing so. Therefore, the top layer of concrete experiences tensile stress, and small shallow cracks are developed in any direction because of the low strength of the concrete at that stage, however, cracks appearing at the time of construction due to shrinkage are usually fine and, though undesirable, may not adversely affect the performance of the bridge deck (NCHRP 1979). Studies have shown that narrow cracks (0.01 in.) have little influence on the overall corrosion of reinforcing steel (NCHRP 1979). Studies have also revealed that it is not the crack width but the total area covered by the cracks, especially close to the steel surface, that is most important for the formation of corrosion (Vaysburd 1993, Mehta and Gerwick 1983). Crack width is normally measured ¼ inch below the surface since the cracks appear wider on the surface because of broken edges. National Cooperative Highway Research Program (NCHRP) pointed out that cracks that follow the line of a reinforcement are more serious, not only because they follow the length of the reinforcement, but they make concrete more susceptible to spalling (NCHRP 1979).

The amount and extent of cracking, to a large degree, depends on the magnitude of drying shrinkage. In the presence of moisture, the concrete tends to expand due to the nature of its constituents. For example, some aggregates absorb water and swell. Upon drying, the reverse takes place. Other factors that influence drying shrinkage include the composition and fineness of the cement, gradation of aggregates, type and shape of the structure, and admixtures. High-early-strength and low-heat cements show more shrinkage than normal Portland cement, and shrinkage is directly proportional to the fineness of the cement (Hassoun 1988, Krauss 1996). Concrete with smaller-sized aggregates also tend to exhibit higher shrinkage. Although all admixtures do not significantly affect shrinkage, those that increase the water requirement of the concrete increase shrinkage; however, the most controllable factor affecting shrinkage is the amount of water per unit volume (Kosmatka and Panarese 1988). Research at MIT showed that for every 1% increase in mixing water, concrete shrinkage increases about 2% (Kosmatka and Panarese 1988). Contrary to Hossoun, Kosmatka pointed out that the type of cement, cement fineness and composition, and cement content have relatively little effect on the drying shrinkage of normal-strength concrete.

Three dominant types of cracks are normally found in the bridge decks: transverse, longitudinal, and diagonal cracks. In addition to these cracks, there are random cracks and pattern or map cracks; however, only the first three types of cracks will be dealt with in this investigation.

The Transverse cracks are the most prevalent (PCA 1970, Pezze and Fu 1992). These cracks usually appear soon after the deck is placed and are often formed directly above or near the transverse bars in the top layer of reinforcement (Schmitt and Darwin 1995, Krauss 1996). Transverse cracks have been found to be more prevalent in continuous span bridges; they seem to occur more frequently with increased span length and age. The supporting structures on the bridges also affect this type of cracking; concrete bridge decks supported on steel girders tend to have a greater amount of transverse cracks (Schmitt and Darwin 1995). These stresses are largely caused by concrete shrinkage and changing bridge temperature, and to a lesser extent by traffic (Krauss 1996). The presence of transverse cracks in bridge deck slabs significantly reduce the longitudinal stiffness of the slab, while the stiffness in the transverse direction is not affected. Transverse cracking also changes the behavior of the deck slab. Normally, the slab behaves like a plate, but after transverse cracking, it starts behaving like a series of individual panels. This type of crack tends to run along the top steel in skewed bridges in which the transverse reinforcing steel has been placed parallel to the skew (Schmitt and Darwin 1995). Although the exact cause of cracks in bridge decks is difficult to pinpoint, the two major factors appear to be the degree of restraint provided by the reinforcing steel and the supporting girders to the early and long-term shrinkage of concrete and the presence of transverse reinforcing steel near the surface that acts as a tensile stress raiser (PCA 1970, Schmitt and Darwin 1995). In addition to these major factors, Krauss (1996) summarized several other major factors affecting cracking in decks that are summarized in Table 2.1. They include the degree of restraint, modulus of elasticity, creep, heat of hydration, aggregate type, cement content and type, weather, and time of casting. One of

Table 2.1 Table showing factors affecting cracking (Krauss 1996)

Factors	**Effect**			
	Major	Moderate	Minor	None
Design				
Restraint	+			
Continuous/simple span		+		
Deck thickness		+		
Girder type		+		
Girder size		+		
Alignment of top and bottom reinforcement bars				
Form type			+	
Concrete cover			+	
Girder spacing			+	
Quantity of reinforcement			+	
Reinforcement bar sizes			+	
Dead-load deflections during casting			+	
Stud spacing			+	
Span length			+	
Bar type – epoxy coated			+	
Skew			+	
Traffic volume				+
Frequency of traffic-induced vibrations				+
Materials				
Modulus of elasticity	+			
Creep	+			
Heat of Hydration	+			
Aggregate type	+			
Cement content and type		+		
Coefficient of thermal expansion		+		
Paste volume-free shrinkage		+		
Water-cement ratio		+		
Shrinkage compensating cement		+		
Silica fume admixture			+	
Early compressive strength			+	
HRWRAs			+	
Accerating admixtures			+	
Retarding admixtures			+	
Aggregate size			+	
Diffusity			+	
Poissoin's Ratio				
Fly ash				+
Air content				+
Slump†				+
Water content				+
Construction				
Weather	+			
Time of casting	+			
Curing period and method		+		

Finishing procedures		+		
Vibration of fresh concrete			+	
Pour length and sequence			+	
Reinforcement ties				+
Construction loads				+
Traffic-induced vibrations				+
Revolution in concrete truck				+

† within typical ranges

the many interesting findings of Krauss' research was that traffic volume had no effect on transverse cracking in the decks.

Longitudinal cracks are normally formed above the top longitudinal reinforcing bars. The most significant contributing factor to this type of cracking is the resistance to volume change that the reinforcing bars impose during the setting stage of the concrete just after placement and finishing (PCA 1970).

Diagonal cracks are the least common among the three types of cracks mentioned (Pezze and Fu 1992). This type of crack is generally found near the ends of skewed bridge decks and over single-column piers (Schmitt and Darwin1995, PCA 1970). The variation of the direction of principal moment along skewed bridge decks may be a possible cause of this type of cracking.

2.1.2 Spalling

Spalling is caused by the deterioration of reinforced concrete as a result of corrosion of embedded reinforcement. Cracks facilitate the movement of corrosive agents, such as chloride ions to the level of the reinforcement, or those substances permeate through the concrete to the steel reinforcement. In poor quality concrete, channels develop from bleed water and speed the ingress of chloride ions (NCHRP 1979). Corrosion in the decks can be detected by the appearance of a brown spot on the surface of the deck that darkens as the corrosion progresses. The oxide formed from the corrosion of the steel bars occupies a much larger volume. Consequently, considerable force is exerted on surrounding concrete. Pressure as high as 4,700 psi has been reported (NCHRP 1979). A corrosion pit of 0.001 inches is sufficient to crack a 0.875-inch-thick concrete cover but is structurally insignificant on the strength of the steel (NCHRP 1979). If the corroding area of the bar is sufficiently large, then a trough or conical spall occurs (NCHRP 1979). Spalling exposes the bars to corrosive agents, and in advance stages, significant pitting of the reinforcement occurs that may considerably affect the structural integrity of the bars. Figure 2.1 shows the possible causes of corrosion and spalling in bridge decks.

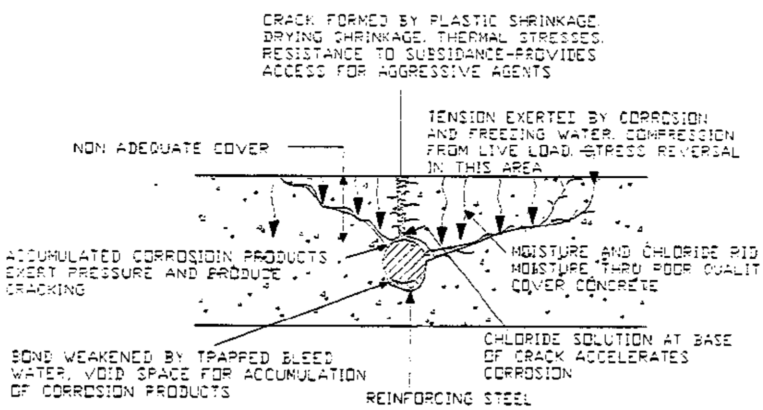


Figure 2.1 Possible causes of corrosion and spalling in bridge deck (Vaysburd 1993)

2.1.3 The Arching Effect

The arching effect is a phenomenon in which a concrete slab that is laterally restrained develops an internal arch when subjected to loads perpendicular to its plane.

The restraints necessary to develop the arching action are provided by two main components, the external girders and the bottom reinforcement (Bakht and Mufti 1996). Figures 2.2 and 2.3 illustrate two concepts of arching effect by Bakht et al. (1996) and Allen (1991), respectively. Bakht (1996) illustrates that this effect develops prior to cracking, but Allen (1991) believes that the effect develops after cracking occurs in the slab; however, both cases show the importance of the lateral stiffness of the upper sections of the girders in the development of the compressive membrane action. Many investigators have supported the theory of arching action after performing a number of tests on a set of models and prototypes. They indicated that the compressive membrane forces significantly increase the flexural capacity of slabs (Girolami 1970, Gamble 1970, Fang et al 1990, Beal 1983, Tsui, Burns, and Klinger 1986, Bakht and Mufti 1990).

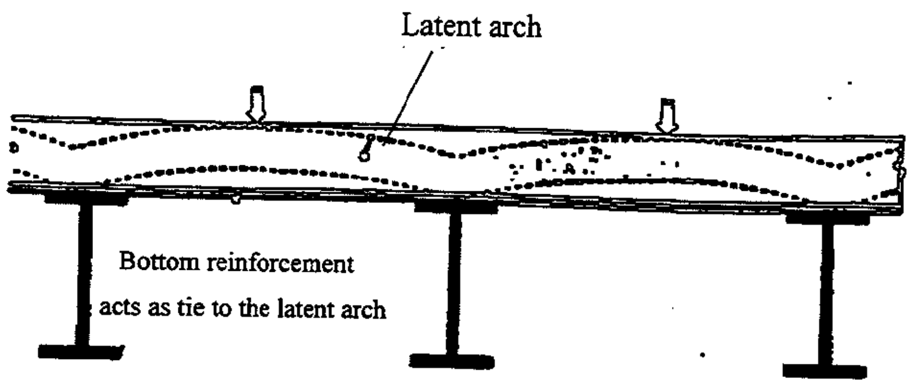


Figure 2.2 Arching action in bridge deck slab (Bakht and Mufti 1996)

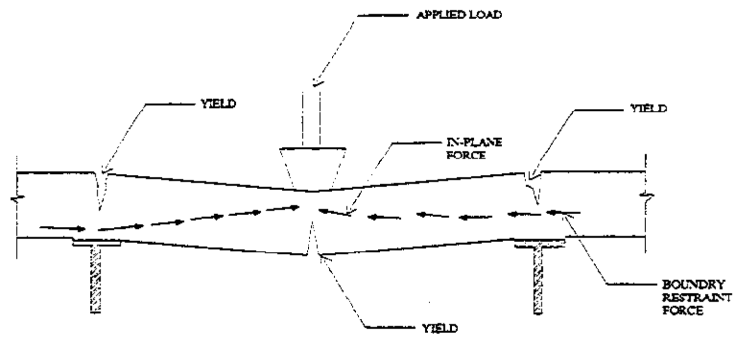


Figure 2.3 Illustration of the arching effect (Allen 1991)

Figure 2.4 shows the relationship between the applied load and compressive membrane action in a concrete deck slab. The membrane force is tensile up to a load of about 50 kips, after which it suddenly goes into compression. This change occurred after cracking in the deck, exemplifying that the compressive membrane action is a post-yield effect (Fang et al 1990). It also shows that the transverse membrane action is completely in compression during the post-fatigue stage. After testing full-scale isotropically reinforced cast-in-place decks in accordance with Texas DOT requirements, however, Fang et al (1990) concluded that the decks perform satisfactory with respect to AASHTO requirements, but, Fang (1990) stated that the membrane action did not have any significant effect on the performance of the decks until there was a lot of cracking in the deck.

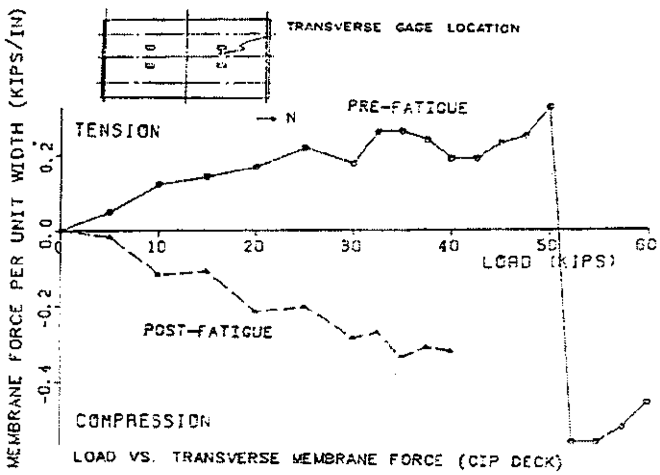


Figure 2.4 Load versus Transverse Membrane Force (Fang et al1990)

2.1.4 Skewed Deck Behavior

A number of bridges are built with some degree of skewness. Skew angle becomes significant when it is greater than 20° in simple supported decks (Figure 2.6). Although the effect of skew in simply supported beams with skew angle less than 20° is negligible, it becomes significant at lower angles in continuous decks, particularly in region of intermediate supports (Hambly 1976).

Skewed bridge decks can pose a significant challenge for design because of several factors. Figure 2.5 shows the characteristic behavior of skewed decks, which exhibit variation of maximum bending moment across the width, from near parallel to span at edge, to near orthogonal to abutment in central regions (Hambly 1976). They also possess negative moments near the obtuse corners. The high twisting moments induced in skewed slabs require them to have extra reinforcements, especially near the supports (Bakht and Agarwal 1995). The magnitude of the torsion depends on the skewness, span width ratio, and, particularly, the type of construction and deck supports (Hambly 1976). The acute corner ends of the decks tend to bend upward, resulting in low reaction and possible uplift, as opposed to the obtuse ends that normally possess high reaction and shear forces. Supporting the decks on soft bearings can reduce the negative effects of skewed decks (Hambly 1976).

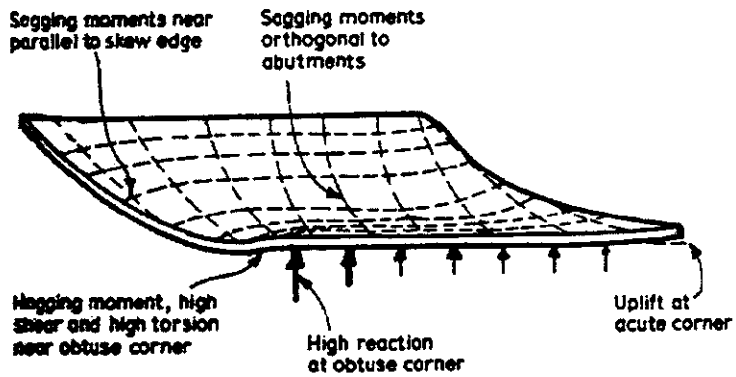


Figure 2.5 Characteristics of skewed deck slab (Hambly 1976)

2.1.5 Laboratory and Field Testing of Isotropic and AASHTO Bridge Decks:

The beneficial effects of the arching action have been utilized in Ontario, Canada, in designing isotropic decks. The Ontario Highway Bridge Design Code (OHBDC) employs an empirical design method for designing bridge deck slabs if certain conditions are met. The code specifies a minimum slab thickness of 9.0 inches, with an additional three eighths of an inch on the surface to provide for wear (OHBDC 1987). According to the OHBDC, the empirical design method may be used if the following conditions are met:

1. There are at least three longitudinal girders placed in the system, and the diaphragms extend throughout the cross section of the bridge between the external girders;

2. The center-to-center spacing of supports for a slab panel measured perpendicular to the direction of traffic should not exceed 12 feet, and the slab should extend at least 3.3 feet beyond the center line of the external longitudinal supports of the panel;

3. The ratio of the center-to-center spacing of supports to thickness of the slab does not exceed 15;

4. The isotropic reinforcement spacing should not exceed 12 inches.

Bridge decks with skew less than 20° should contain isotropic reinforcement of up to 0.3% of the gross cross sectional area; however, when the skew angle is greater than 20° the exterior region is provided with reinforcement of up to 0.6% of the gross cross sectional area, as illustrated in Figure 2.6. The reason for this increase reinforcement ratio is that the highly skewed section of the deck has a significantly lower moment capacity. In the case where the reinforcements are placed parallel to the abutments, the reinforcement ratio shall not be less than $0.003/\cos^2(\theta)$in interior region and $0.006/\cos^2(\theta)$in the exterior region, where θ is the skew angle (OHBDC 1987). For example, θ in Figure 2.6 is 20°. These conditions are set in order to reduce the incidence of cracking in the skewed regions of the decks.

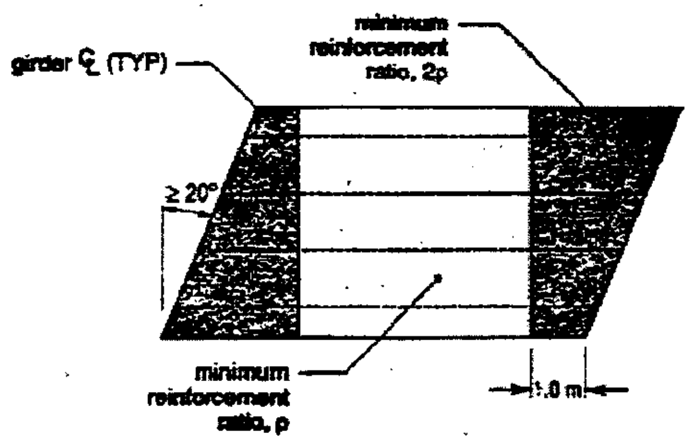


Figure 2.6 OHBDC reinforcement pattern for skewed deck bridges (Jaeger et al 1995)

The New York Department of Transportation (NYDOT) has compiled an extensive report on the comparative performance of isotropic and AASHTO-type bridge deck slabs and have reported some interesting findings. The decks were compared based on their age, structural type, and traffic volume. All the bridge decks investigated by NYDOT were supported on steel girders. It was observed that the average transverse crack densities of the AASHTO and isotropic decks were 1.37in/sq. yard and 1.61in/sq. yards, respectively. This is not considered to be a substantial variation (Pezze III and Fu 1992; however, there was a significant difference between the longitudinal crack densities. The average longitudinal crack densities were 1.04 in/sq. yard and 0.26 in/sq. yards for isotropic and AASHTO decks, respectively. The report concluded that isotropic decks performed well, with no signs of spalling, delamination, or severe cracking. One of the conclusions of the report was that cracking occurrence was also affected by the other factors. In particular, construction operations were considered to be significant. It was also reported that higher crack densities were observed on decks with removable forms compared to those with stay-in-place forms. Another observation was the effect of pouring procedure on cracking; transverse crack densities could vary significantly from pour to pour (Pezze III and Fu 1992). Figures 2.7 to 2.9 show some of the findings of the investigation done by Pezze III and Fu.

Figure 2.7 shows the percentage of total crack density found at the isotropic sites. It clearly indicates that transverse cracking is the most prevalent, followed by longitudinal cracks; however, the occurrence of diagonal cracks was relatively very low. The similar graph (Figure 2.8) for the AASHTO sites shows that transverse cracks were by far the predominant cracks found in AASHTO decks; however, they show a relatively smaller amount of longitudinal cracks than isotropic decks, but both decks exhibited the same percentage of diagonal cracks.

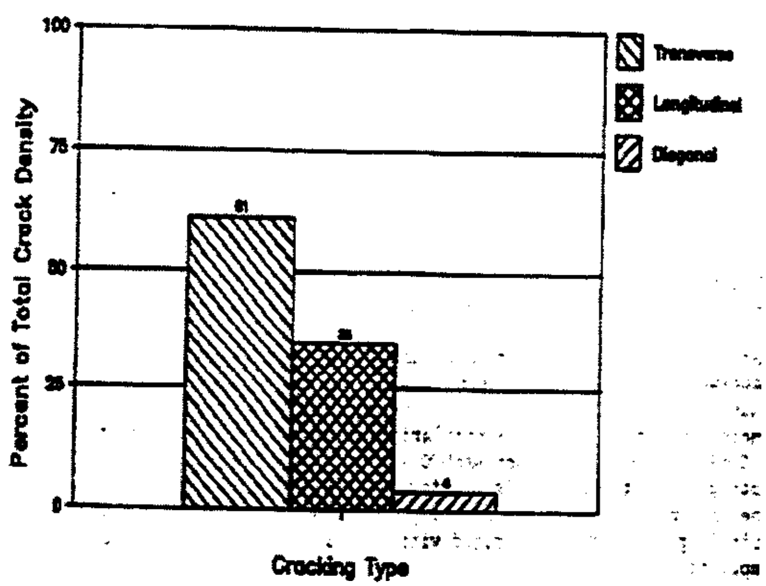


Figure 2.7 Total crack lengths at isotropic sites (Pezze and Fu 1992)

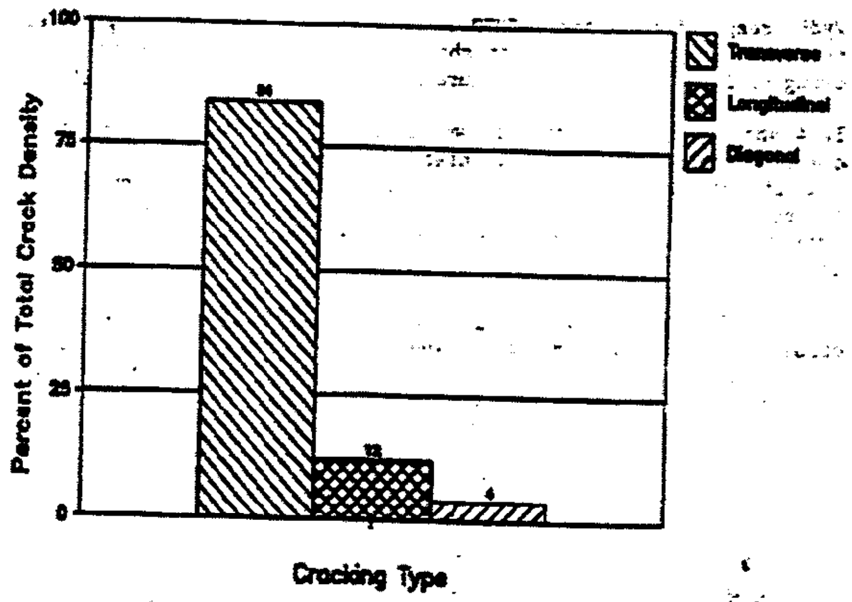


Figure 2.8 Total crack lengths at AASHTO sites (Pezze III and Fu 1992)

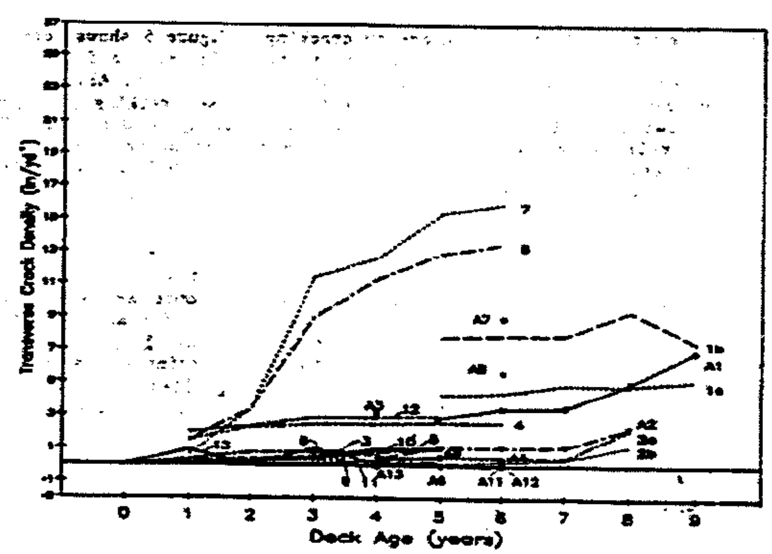


Figure 2.9 Transverse crack density on deck top surfaces (Pezze III and Fu 1992)

Figure 2.9 shows a plot of the transverse crack density on the top surfaces of the AASHTO and isotropic decks. The number preceded by the 'A' represents AASHTO decks, while the others represent different isotropic decks. Generally, there was not a considerable difference between the two types of decks. Sites 7, 8, A7, and A8 were located along the same route, but decks that showed considerably higher crack density compared to the other decks were considered to be anomalies in the investigation. Consequently, they were left out of the general comparison. The reason for the relatively high amount of cracking was attributed to construction procedures, together with overloads in the cases of 7 and 8.

The Michigan Department of Transportation is also investigating isotropic decks. The department constructed two bridges, each of which had sections of isotropic and AASHTO reinforcements. They reported that the isotropic and AASHTO decks exhibited similar crack patterns; however, the crack density in the AASHTO deck was slightly higher. The difference was attributed to variation in construction. The isotropic reinforcement on one bridge used approximately fifty percent of the conventional AASHTO design. Savings of approximately $186,000 in steel cost over the conventional method would result if the deck were constructed completely of isotropic reinforcement (Reincke 1997). This is a savings of approximately $21.80 per square foot of slab.

Another interesting aspect of the bridge deck was the location of crack formations. It was found that the service load moment over the girders was significantly below the cracking moment of the slabs; however, the positive moment was near that of the cracking moment in the positive moment region. Consequently, cracks were more prevalent on the underside of the decks, as illustrated in Figure 2.10 (Allen 1991). A number of isotropic decks observed also exhibited extensive longitudinal cracking that implied that they experienced flexural cracking (Allen 1991). This observation was made on decks supported by I-beams, and according to Allen, this observation may not apply to decks supported on stiff members, such as concrete and steel box girders. He further described two limit states with respect to the strength of bridge decks, yield mechanism and collapse mechanism, as illustrated in Figure 2.10 and Figure 2.11, respectively. The yield mechanism is first formed in the decks where the positive moment yield load is reached. This mechanism indicates that the negative moment is much less than the positive moment in the slab. This was attributed to the inherent arching action; however, the compressive membrane does not occur until hinge formation occurs below the load and over the girders. This gives rise to the collapse mechanism, which is a full flexural mechanism in both the positive and negative moment regions. The collapse strength is enhanced by higher volumes of reinforcement in the bottom layer of the slab, justifying why Allen favored conventional AASHTO decks over isotropic decks.

Since the present practice in the design of isotropic decks only employs a percentage of steel significantly below the AASHTO specification, these decks do not meet the AASHTO requirement for flexural design. Hence, these decks should be designed so that they do not experience flexural cracking. Once flexural cracking occurs, these lightly reinforced concrete decks do not have enough strength and are likely to fail. Consequently, the cracking strength of the deck slab is a crucial factor in the design of lightly isotropic deck slabs. Allen also observed that the isotropic bridge decks in Canada did not show extensive flexural cracking as those that he'd observed in the United States. He attributed the difference to the variation in slab thickness; the Ontario isotropic bridge decks were thicker than those under investigation in the U.S. Calculations show that a small increase in the thickness of the decks can result in significantly higher cracking moments.

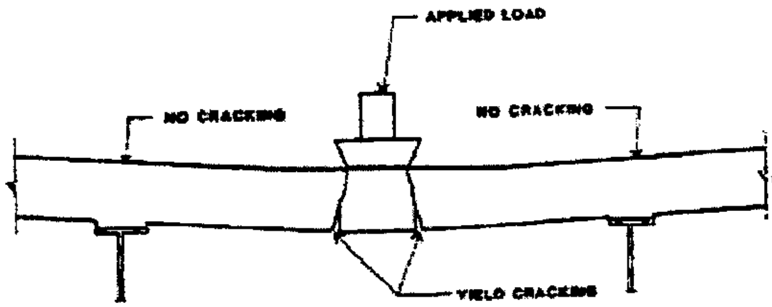


Figure 2.10 The yield mechanism (Allen 1991)

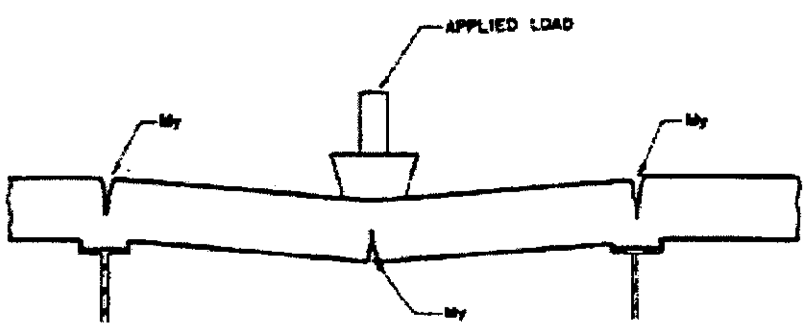


Figure 2.11 Collapse mechanism (Allen 1991)

The following formulas can be used to calculate the cracking strength moment of the concrete:

$$M_{cr} = f_r t^2/6 \qquad (2.1)$$

Where:

t = thickness

f_r = modulus of rupture

Newmark (1949):

$$M_{cr} = P_w/(3+10b/c) + P_w(L-60)/1000 \quad \text{(for spans over 60ft)} \tag{2.2}$$

For spans up to 60 feet, Newmark proposed the following formula:

$$M_{cr} = P_w/(3+10b/c) \tag{2.3}$$

Where:

P_w = wheel load

L = span length

b = beam spacing

c = wheel width

Beal (1983) confirmed some of the above behavior of bridge decks in which he used isotropic and AASHTO reinforcement patterns. Scaled models were used in the laboratory, and full-scale decks were also evaluated in the field. His isotropic decks full-scale consisted of #4 bars spaced 12 inches in the transverse and longitudinal directions with deck thickness of 8 inches. The decks failed in punching shear at wheel loads exceeding 130 kips. Some of the tested models used by Beal are shown in Figure 2.12; they all show punching shear failure. A 70-kip load criteria was set as the service strength limit of the slabs where the live wheel load was 16 kips. Therefore, slabs having an average punching shear strength greater than or equal to 70 kips were considered to have adequate strength for the bridge decks.

Perdikaris and Bein (1988) have carried out tests on 3/5-scale models to investigate the fatigue behavior of AASHTO decks and isotropic decks under moving loads and stationary pulsating loads. The isotropic deck was designed in accordance with the OHBDC. A third type of slab was designed with no reinforcement. The isotropic deck was reinforced with 0.3% steel in the top and bottom transverse and longitudinal directions, while the AASHTO-type deck was reinforced with 0.7% and 0.35% in top and bottom, both in transverse and longitudinal directions, respectively. They also observed that the unreinforced deck failed in flexure, while the isotropic and orthotropic failed in punching shear. They concluded that the AASHTO deck had a higher fatigue life than the isotropic deck when both were subjected to fixed pulsating loads; however, the isotropic deck had a fatigue life of up to 20 times that of the orthotropic deck when it was subjected to moving loads. The latter appears to give decks an advantage

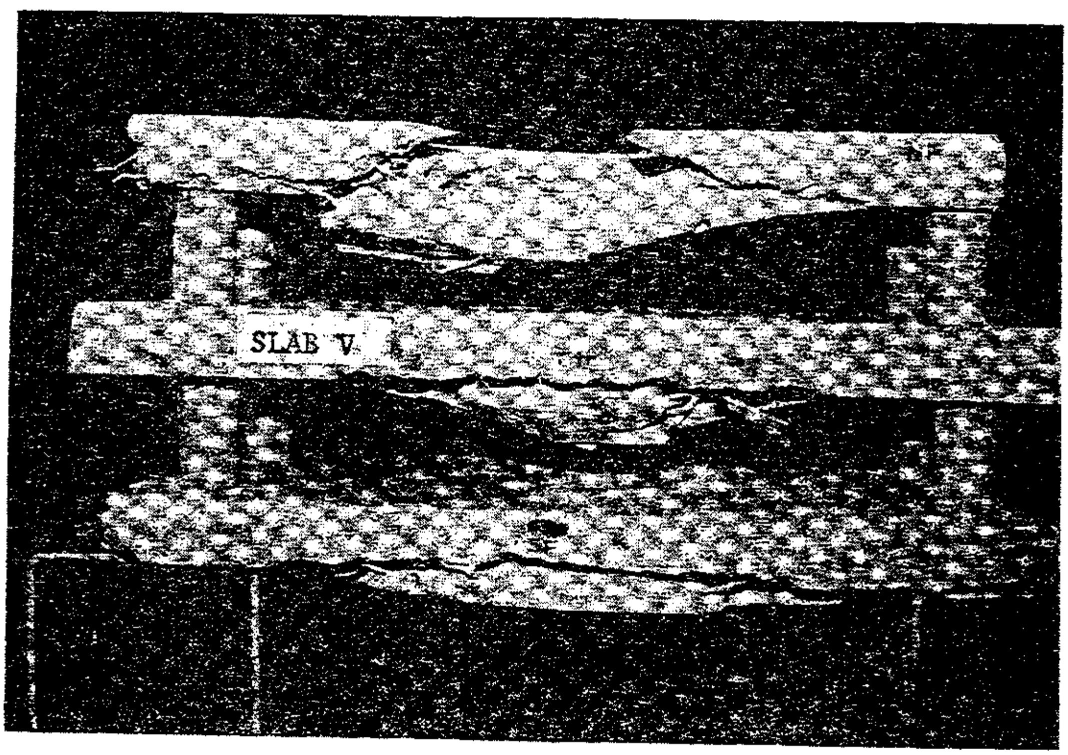


Figure 2.12 Punching shear failure (Beal 1983)

over its orthotropic counterpart with respect to fatigue criteria; although the orthotropic deck endured longer under the pulsating load, the moving load is more realistic regarding what the bridge will experience in the field.

Crack pattern in the orthotropic deck was more localized under the pulsating load than in the isotropic decks (Perdikaris and Bein 1988); however, cracks were also more extensive in AASHTO-type decks due to the moving loads. They also showed orthogonal crack pattern approximately under the steel reinforcement, but the mode of failure was the same—punching shear. Figures 2.13 and 2.14 show further characteristic patterns in isotropic and AASHTO model decks observed by Perdikaris and Bein. Figure 2.13 shows

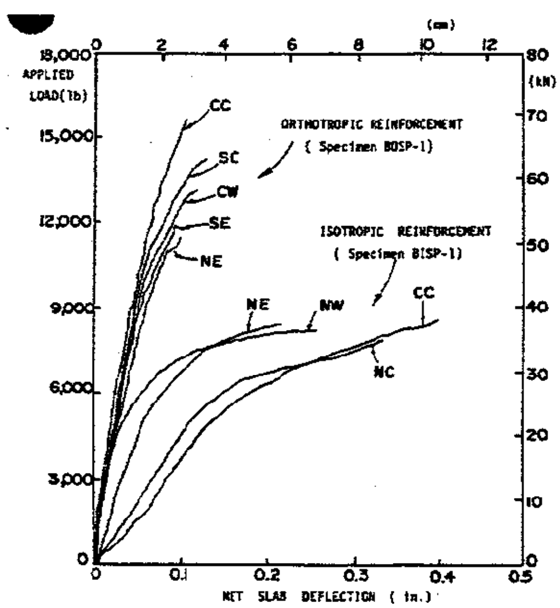


Figure 2.13 Behavior of model bridge decks under static concentrated load (Perdikaris et al 1988)

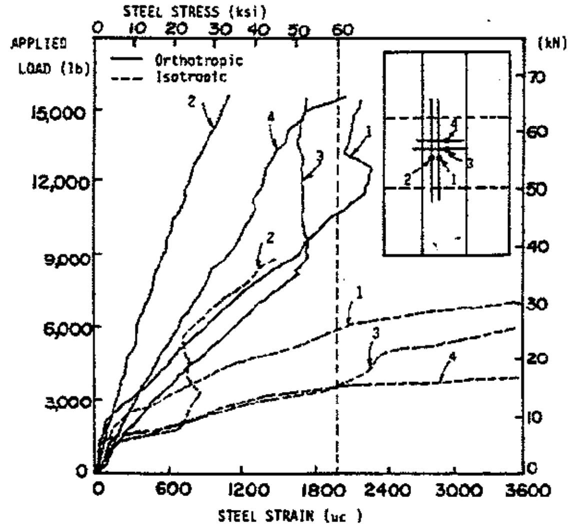


Figure 2.14 Steel stress and strain measurements in bottom reinforcement layer under static load (Perdikaris et al 1988)

that the isotropic deck deflected considerably more than its AASHTO counterpart. This may be attributed to the lower stiffness in the isotropic decks due to lower amounts of reinforcement. The higher amount of steel in the bottom of the section of the orthotropic deck could also have increased the lateral restraint on the deck, thus enhancing the arching effect. Enhancement due to arching could help reduce deflection.

Higher steel strain was observed in the transverse direction of the isotropic deck compared to the orthotropic deck, as was expected because of the lower percentage of steel. It was believed that a higher amount of reinforcements in the transverse direction stiffened the decks in that direction, resulting in higher transverse bending moments and usually higher steel strain; however, the cracking load level was about four times the AASHTO design load in the prototype.

2.1.6 Punching Shear:

Because both the isotropic decks and AASHTO decks failed in punching shear, it is crucial to get some insight into the factors affecting this mode of failure. One variable that affects punching shear strength is the quantity $\sqrt{f_c'}$, where f_c' is the compressive cylinder strength of the concrete. This is because the tensile strength of the concrete is proportional to $\sqrt{f_c'}$, and shear failures are primarily controlled by concrete tensile strength. The second factor that affects the punching shear strength is the ratio of the side length of the loaded area to the effective depth of the slab (c/d). This is illustrated in the following formulas:

$$V_c = 4(d/c + 1)\sqrt{f_c'}\, bd \tag{2.4}$$

$$V_c = 4\sqrt{f_c'}\, b_o d \tag{2.5}$$

Finally, the shear strength is affected by the concrete aggregate. For example, for the same compressive strength, lightweight concrete has a lower splitting tensile strength than that of normal weight concrete (Park and Gamble 2000). Hence, there may be variations in the shear strength of the concrete slabs on different decks or between different batches of concrete. but, it is possible to design concrete mix so that strengths of different mixes will be close in magnitude.

Tsui et al (1986) carried out a number of tests on models and prototype of isotropic decks and reported that the punching shear failure was not the typical 45° shear failure, but the angle of failure was significantly less. They found that the failure angle was approximately 39°. Figure 2.15 shows the assumed failure surface of the general punching shear model. They tested several analytical models and tried correlating them to the actual failure strength. It was found that the yield-line theory including arching action where one-way and two-way action was assumed gave considerably higher shear capacities than the experimental values; however, both the AASHTO and ACI formulas indicated below are conservative in predicting the shear capacity. For example, the actual punching shear failure of one experimental deck was 142 kips, but, the ACI formula predicted a failure load of 104 kips, and the AASHTO analytical value was 46 kips.

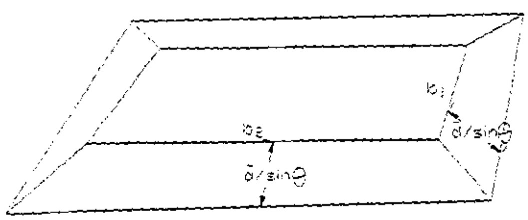


Figure 2.15 Assumed shear surface of general punching shear model (Burns, 1986)

ACI formula:

$$V_c = 2(2+4/\beta_c)(b_1+b_2+2d)\sqrt{f_c'}d < 8(b_1 + b_2+2d)\sqrt{f_c'}d \qquad (2.6)$$

AASHTO formula:

$$V_c = 2(0.8+2/\beta_c)(b_1+b_2+2d)d\sqrt{f_c'} \qquad (2.7)$$

β_c is the ratio of b_2 to b_1

d = the average effective depth of the section under consideration

Batchelor and Hewitt (1976) also carried out similar tests on isotropic and AASHTO-type decks, confirming the conservatism of the conventional design. After testing a number of isotropic and AASHTO-type models, they reported that the

AASHTO prototype would have a factor of safety of 17 against punching of the deck by a standard HS 20-44 wheel load. They also reported that transverse cracking had negligible effects on the punching strength of the deck and that 0.2 percent isotropic reinforcement is adequate for a conventional 7-inch thick deck slab. In addition, they found that unreinforced panels of conventional dimensions had a factor of safety of 13 against punching shear.

2.2 Review of PENNDOT Documents

2.2.1 List of Bridges and Their Locations

Pennsylvania Department of Transportation carried out their investigation in six engineering districts, including Districts 1-0, 2-0, 3-0, 4-0, 5-0, 9-0, and 10-0. Tables 2.2 and 2.3 show a list of the bridges, including their location by districts and state route, and their location is also shown graphically in Figure 2.16.

Table 2.2 PENNDOT Isotropic Bridge Decks

Engineering District	County	SR	Section	S-Number or Station
1-0	Vernango	62	B01	S-18115
3-0	Union	45	002	S-18417
4-0	Susquehanna	11	572	S-18165
4-0	Wayne	371	670	S-18195
5-0	Berks	2031	01B	S18193
9-0	Blair	4027	003	Sta. 149+57
10-0	Butler	422	252	S-17824

Table 2.3 PENNDOT AASHTO Designed Bridge Decks

Engineering District	County	SR	Section	Structure No. or Station
1-0	No AASHTO designed counterpart			
3-0	Lycoming	973	390	S-17724
4-0	Wyoming	11	770	S-17938
4-0	Wayne	170	670	S-18057
5-0	Berks	82	05B	Sta. 335+00
9-0	Somerset	403	16	S-18251
10-0	Butler	422	251	S-17983

Key: ● Isotropic decks ●AASHTO deck locations

Figure 2.16 Location of AASHTO and Isotropic bridge decks

2.3 District 1-0

One experimental bridge deck was constructed in District 1-0. The deck was located in Venango County over Conrail and French Creek on SR 62 (Figure 2.17) and was constructed in June 1991. A section of the deck during construction is shown in Figure 2.18. No AASHTO control deck was constructed for this isotropic deck. The bridge had a total of five spans, with one simple span 61 feet and 6 inches in length. The rest of the deck consists of a four-span continuous system with lengths 124 feet, 125 feet, 117 feet-6 inches, and 124 feet, resulting in a total deck length of 554ft-0 in. It supports four

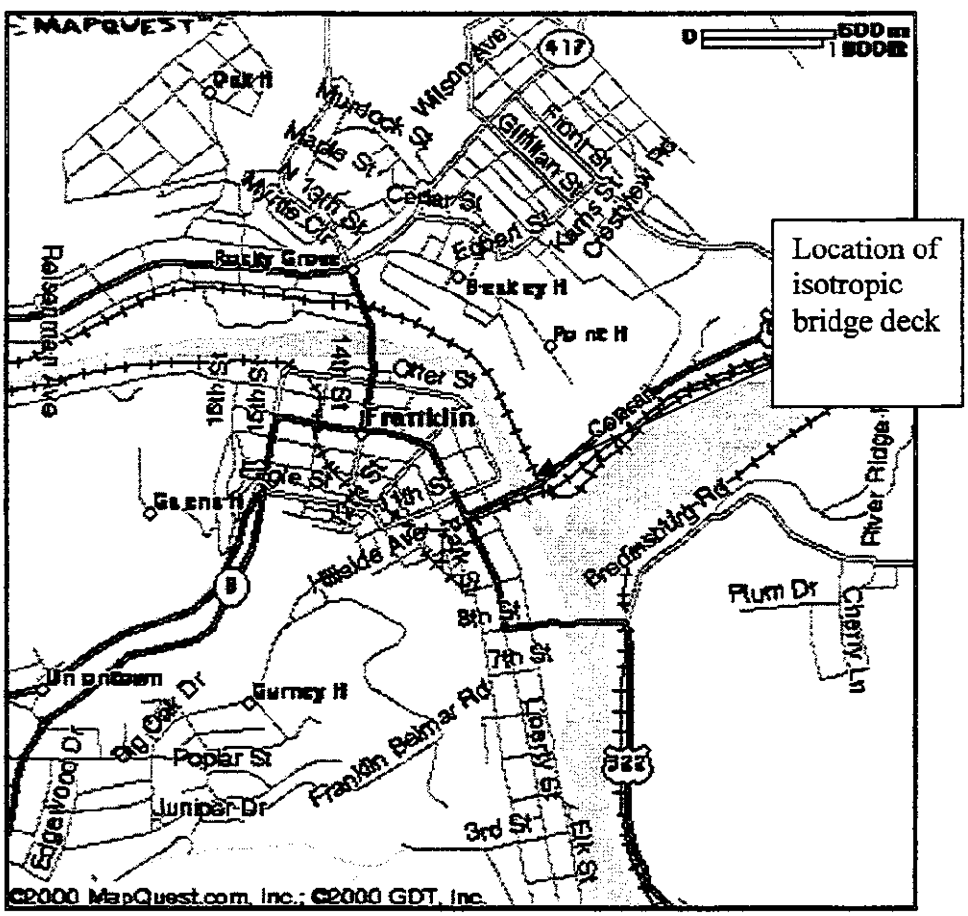


Figure 2.17 Location map showing of the structure on SR 62

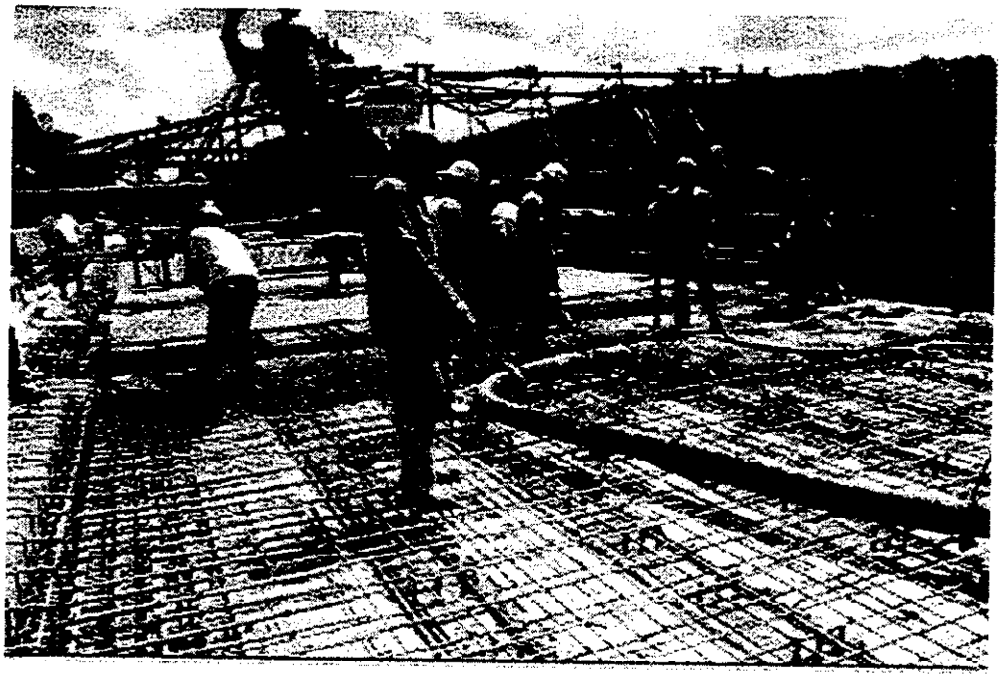


Figure 2.18 Construction of SR 62 isotropic deck

lanes of traffic, two in each direction, and a section was supported by seventeen w33x130 steel beams spaced 3feet-1 inch on center. The road width is 26 feet-6inches, and 63 feet out to out, and the deck was skewed at 90°. A summary of the general properties of the deck including results from field visits is shown in Table 2.4.

Reinforcement in the positive moment region consisted of #4 bars placed 12 inches in the top and bottom transverse and longitudinal direction. Number 5 bars were placed 6 inches center-to-center in the top transverse and longitudinal directions in the negative moment regions, and #4 and #5 bars were place transversely and longitudinally in the bottom mat, respectively.

Preliminary cost analysis of the deck estimated that a standard 8-inch-thick deck showed the cost \$463,212, while the isotropic deck would cost \$428,491 for an 8.5-inch-thick deck. Hence, the use of isotropic deck resulted in approximately \$35,000.00 in savings. The deck was approximately 39,902 sq. ft, which implies that there was a savings of $1.00/sq. ft. of slab.

Table 2.4 District 1-0 bridge deck summary

County		Vernango		(no AASHTO counterpart)	
District		1-0			
SR Number		62			
Number of Spans		5			
Span Lengths		61'-6",124'-0", 125'-0", 117'-6", 124'-0"			
Width		63' (out-to-out)			
Type of Deck		Isotropic			
Skew (degrees)		90			
Girder spacing		3'-1 1/4"			
Completion of Construction		June 1991			
Material Cost/ And Labor cost		$2.67ft² (positive reinforcement) 4 units of labor			
Total Crack Length	**Transverse**	Year 1	63	Year 1	
		Year 3	96	Year 3	
		Year 5	N/A	Year 5	
	Longitudinal	Year 1	0	Year 1	
		Year 3	104	Year 3	
		Year 5	N/A	Year 5	
	Diagonal	Year 1	0	Year 1	
		Year 3	0	Year 3	
		Year 5	N/A	Year 5	
Number of Permit Loads		None			
Miscellaneous		S # : 18115 Area= 3878 sq. yds			

Inspection done in December 1991 showed that the cracks of low severity were detected and that no cracks were detected in the northbound lane. In August 1994, the field inspection carried out indicated that there were only minor cracks. Their locations were sketched on the bridge schematic diagram. In June 2000 another field inspection was also carried out in which the surface of the deck was observed for cracking, spalling, and any other significant damages. It was found that the cracks were only of low severity, and there was no occurrence of spalling. Generally, the decks appeared to be performing well. It should be noted that the latter investigation was carried out nine years after the deck was constructed. Hence, this field visit was beyond the five-year period that the data had to be collected; however, it served as useful qualitative information about the performance of the deck. Figures 2.19 to 2.20 shows the condition of the deck during the field visits in 1994 and 2000. The orange marks in the photograph of the deck in 2000 show location of cracks along the deck.

Figure 2.19 Isotropic deck in Venango during August 1994

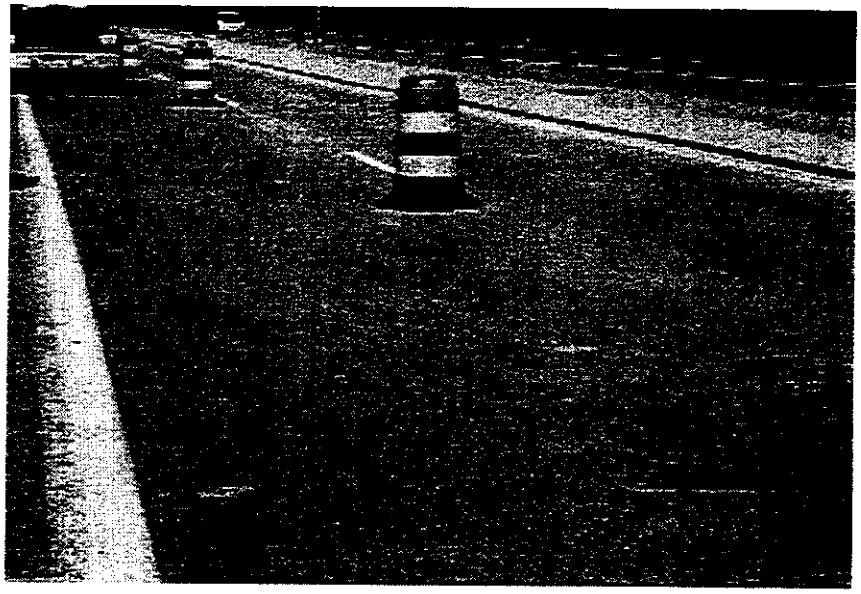


Figure 2.20 Isotropic deck in Venango during August 1994

2.4 District 3-0

Two bridge decks were constructed in District 3-0 for the investigation of isotropic deck performance. The isotropic deck was located along SR 45 in Hartley Township over Spruce Run, Union County (Figure 2.21), while its ASSHTO counterpart was located along SR 973 over Mill Creek in Hepburn Township, Lycoming County (Figure 2.22). The AASHTO deck was constructed in August 1990, and its isotropic counterpart was constructed in October 1991.

The isotropic deck was a simple single span with a span of 54 feet-10 inches and width of 46 feet-6 inches. It supported two lanes of traffic and had two 10-ft shoulders. The average thickness of the deck was 8.5 inches, and it rested upon five prestressed 36-inch x 36-inch concrete box girders spaced 9.44 feet center to center. Its AASHTO counterpart was also a simple single

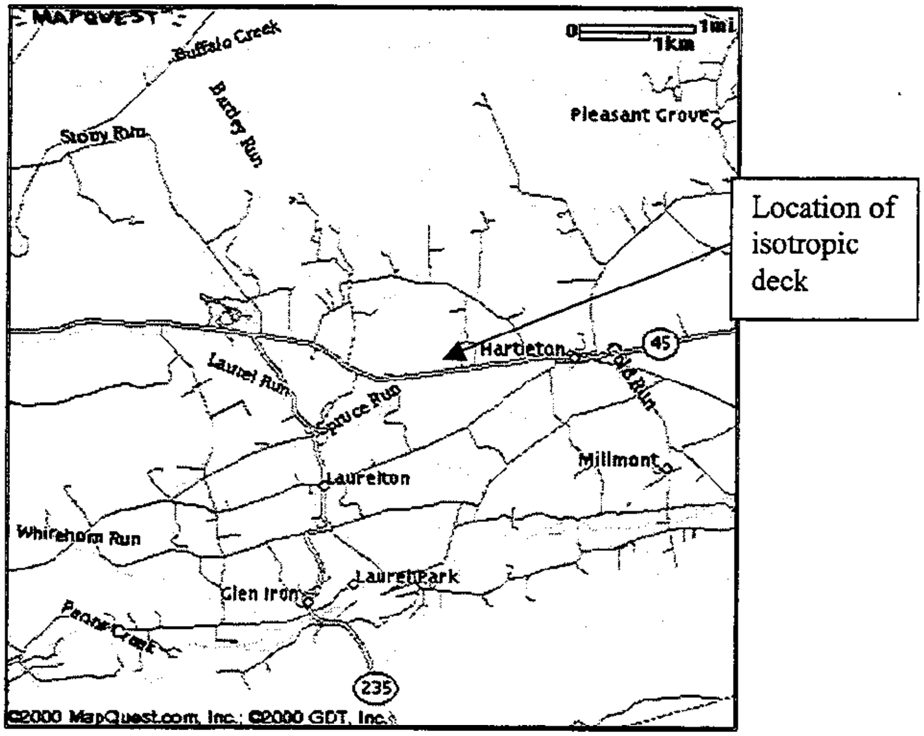


Figure 2.21 Location of Isotropic deck in Union County along SR 45

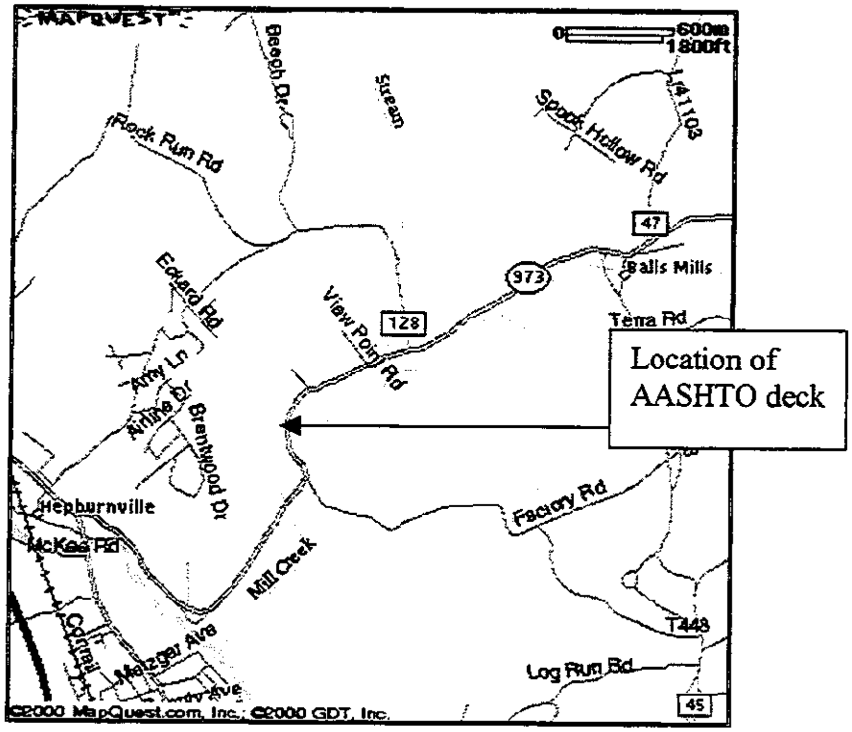


Figure 2.22 Location of AASHTO bridge deck on SR 973 in Lycoming County

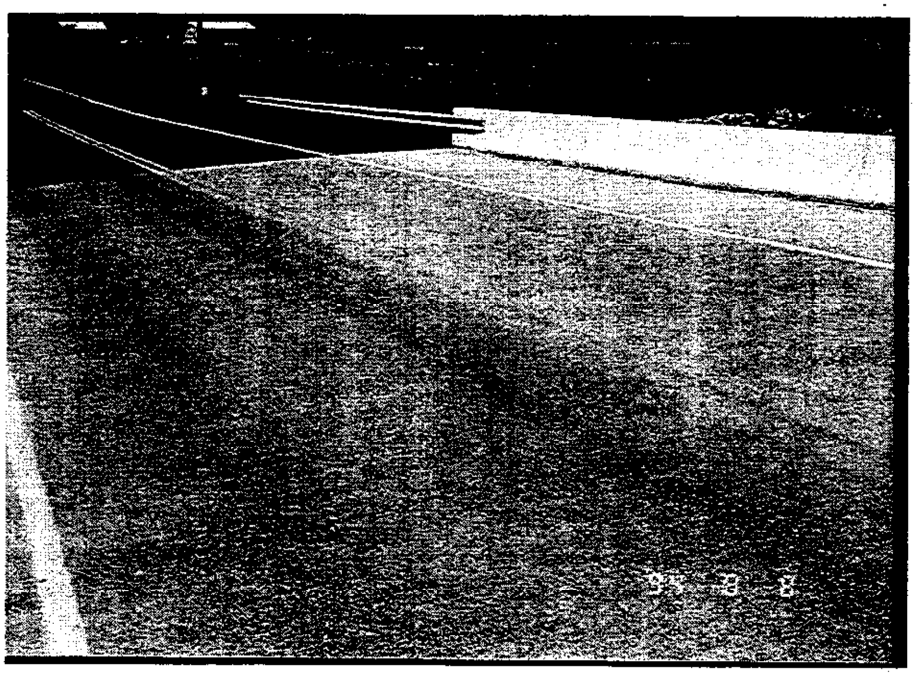
span bridge with clear span of 46 feet and width 41 feet-6 inches, and it rested upon six 27-inch x 48-inch. prestressed concrete spread box girders equally spaced 6 feet-11.5 inches center to center. This bridge also supported two lanes of traffic that were bounded by two six feet shoulders.

The following permit loads were recorded crossing SR 45 between August 1994 and July 1995, for a total of eight times. The type of load was P&H T-750 four-wheel hydraulic truck crane by Allison, Inc., with a GVW 101,700 lbs. Further details of the axles are shown in Table 2.5. No permit loads were recorded on SR 973 during the same period.

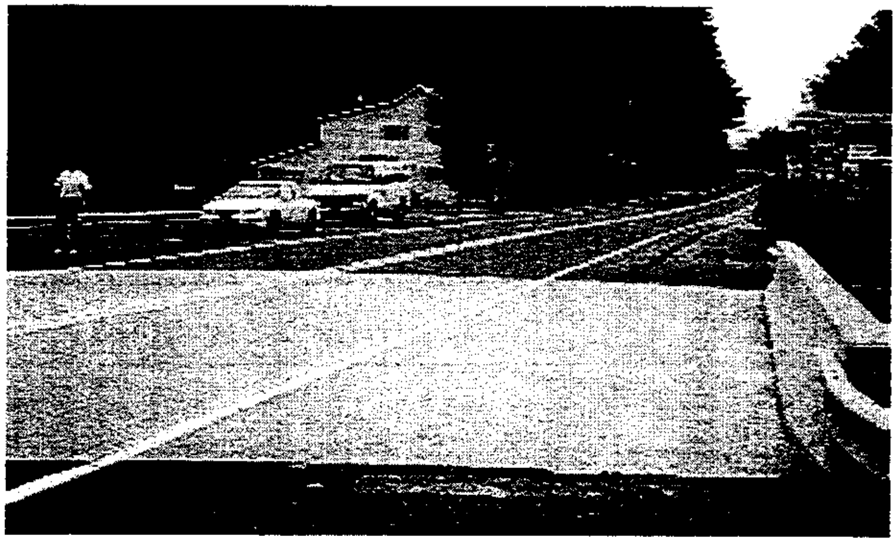
Table 2.5 Description of permit load on SR 45

Axle NO.	Weight (lbs)	Distance from Axle
1	22 350	-----------
2	22 350	4'-6"
3	28 500	14'-1"
4	28 500	4'-6"

Field visits revealed no visible cracks in the isotropic bridge deck. The follow-up visit in June 2000 revealed no visible cracks on the deck. At the time of visit, the bridge was frequently traveled by heavy trucks, as well as lighter vehicles. Figures 2.23 to 2.24 show the appearance of the isotropic deck and AASHTO during the 1994 and 2000 field inspection, and Table 2.6 summarizes some general properties of the decks, including



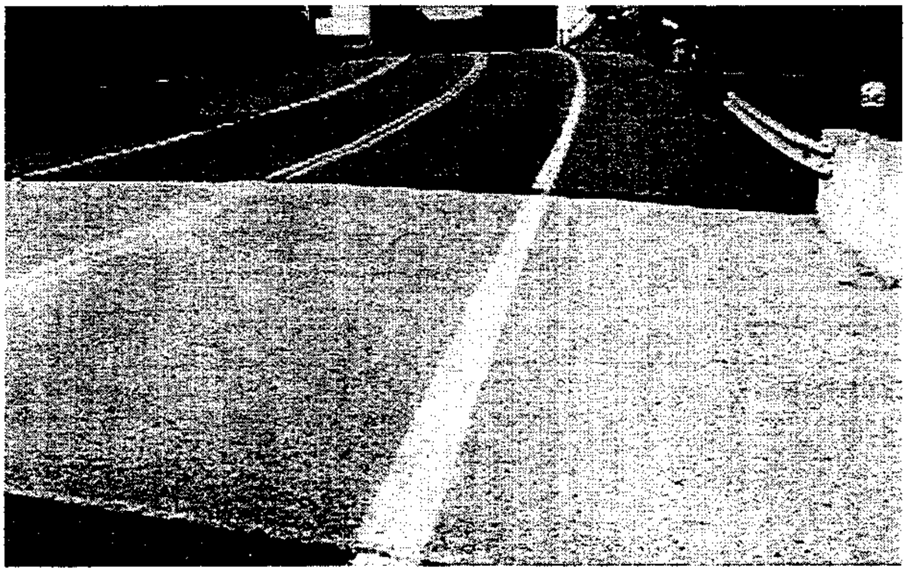
(a)



(b)

Figure 2.23 Isotropic Bridge SR 45, (a) August 1994, (b) June, 2000

(a)



(b)

Figure 2.24 Bridge on SR 973, (a) August, 1994, (b) June, 2000

Table 2.6 District 3-0 bridge deck summary

County	Union	Lycoming
District	3-0	3-0
SR Number	45	973
Number of Spans	1	1
Span Lengths	52'8"	46'0"
Width	47'6"	41'6"
Type of Deck	Isotropic	AASHTO
Skew (degrees)	65	60
Girder spacing	9.44'	6'-11.5"
Completion of Construction	October 1991	August 1990
Material Cost/ And Labor cost	\$2.67ft^2 (positive reinforcement)/ 4 units of labor	\$2.92/ft^2 (for positive reinforcement)/ 3.92 labor units

Total Crack Length	**Transverse**	Year 1	0	Year 1	0
		Year 3	0	Year 3	0
		Year 5	N/a	Year 5	N/a
	Longitudinal	Year 1	0	Year 1	N/a
		Year 3	0	Year 3	0
		Year 5	N/a	Year 5	N/a
	Diagonal	Year 1	0	Year 1	0
		Year 3	0	Year 3	0
		Year 5	N/a	Year 5	N/a
Number of Permit Loads		8		None	
Miscellaneous		Girder size: 36"x36" Type: Prestressed concrete spread box beam Structure #: S-18417 Area: 278 sq. yds		Girder size: 27"x48" Bridge Type: Prestressed concrete spread box beam Structure #: S-17724 Area: 212 sq. yds	

some field data. A number of 4-inch cores were taken from the isotropic deck; however, there were no results of the test of those cores available from PENNDOT documents. The AASHTO deck showed a low occurrence of cracks, and they were of low severity; however, there was a small amount of flaking of the aggregate on portions of the AASHTO deck, but it was not severe and did not appear to have any significant effect on the performance of the deck. Both decks remained in very good condition over the years.

2.5 District 4-0:

A total of four bridge decks were located in this district. The isotropic decks were located along SR 11 and SR 371 (Figures 2.25-2.26) at Susquehanna and Wayne County, respectively. AASHTO counterparts were located along SR 11 in Wyoming County and SR 170 in Wayne County, respectively.

The isotropic deck at Susquehanna County was supported by prestressed box beam, with one span of length 75 feet-6 inches, and skewed 90 degrees. It had a total width of 43 feet-6 inches, out to out.

The AASHTO deck has a total of four spans supported on four 28-inch x 96 inch prestressed concrete I-beam. The first span on either side of the deck is simple supported, while the center is a two-span continuous system. This deck had a total length of 480 feet-2 inches, with span lengths 100 feet-3 inches, 140 feet, 140 feet, and 100 feet- 3 inches.

The AASTHO control deck, located on SR 170, was a simple, one-span deck with a total length of 40 feet and width of 31feet-6 inches out to out. The deck was supported by four 48-inch x 21-inch prestressed concrete box beams equally spaced at 7 feet-10 inches, and has a 90-degree skew.

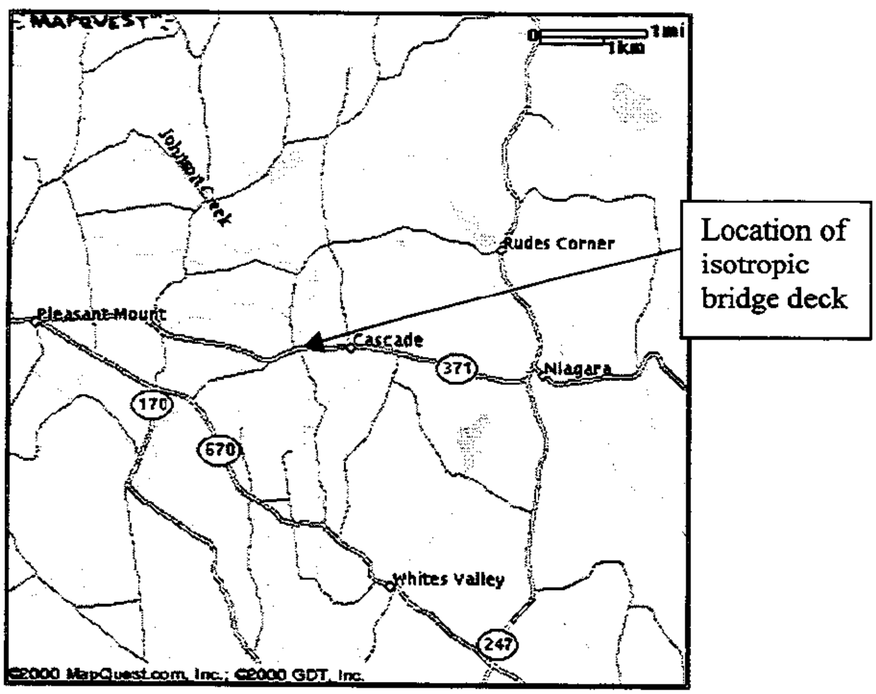


Figure 2.25 Location of structure in Wayne County along SR 371

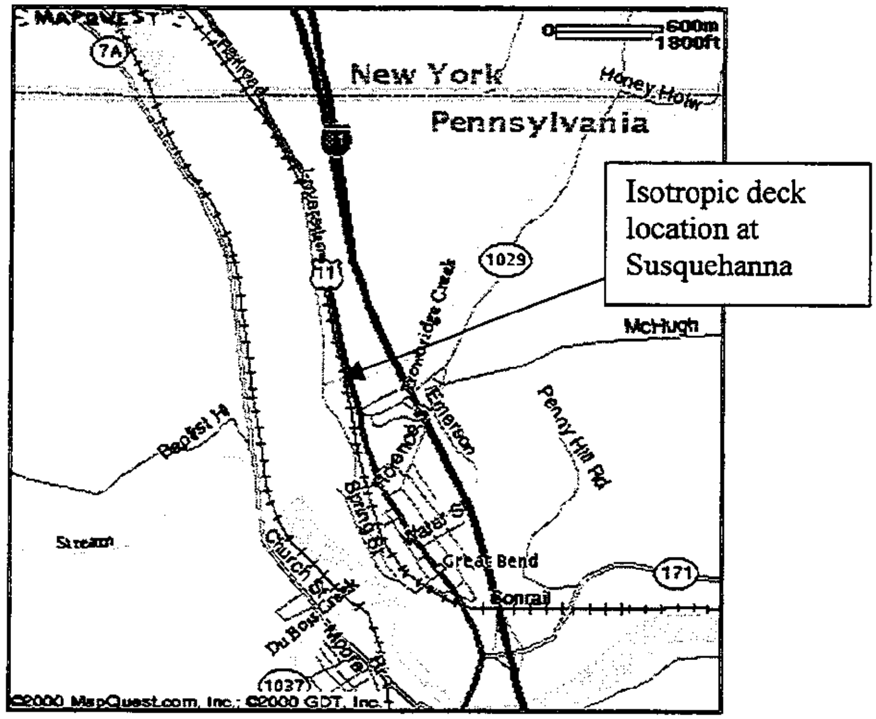


Figure 2.26 Location of isotropic bridge along SR 11 in Susquehanna County

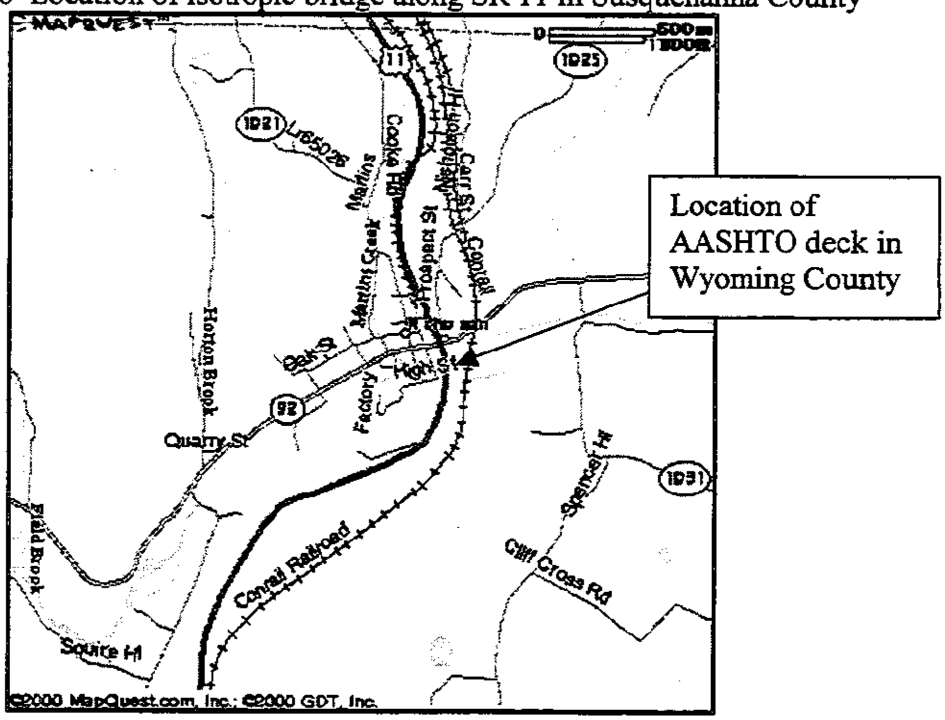


Figure 2.27 Location of AASHTO along SR 11 in Wyoming County

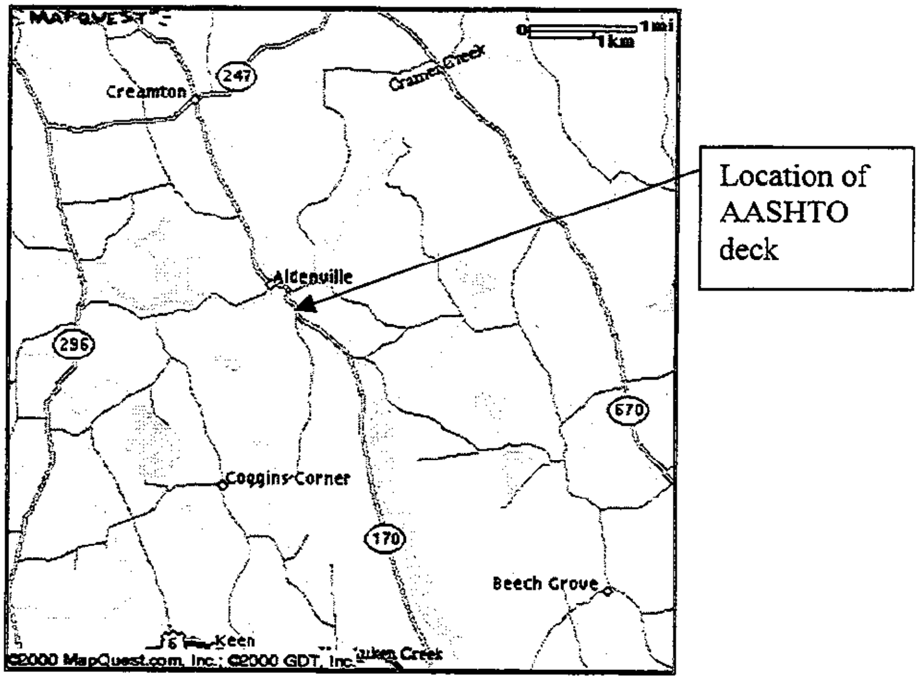


Figure 2.28 Structure location along SR 170 in Wayne County

A field inspection conducted on August 11, 1994, indicated that there were only minor cracks in the isotropic decks at Susquehanna County and Wayne County (Figure 2.29-2.30). Tables 2.7-2.8 summarizes general properties of the decks.

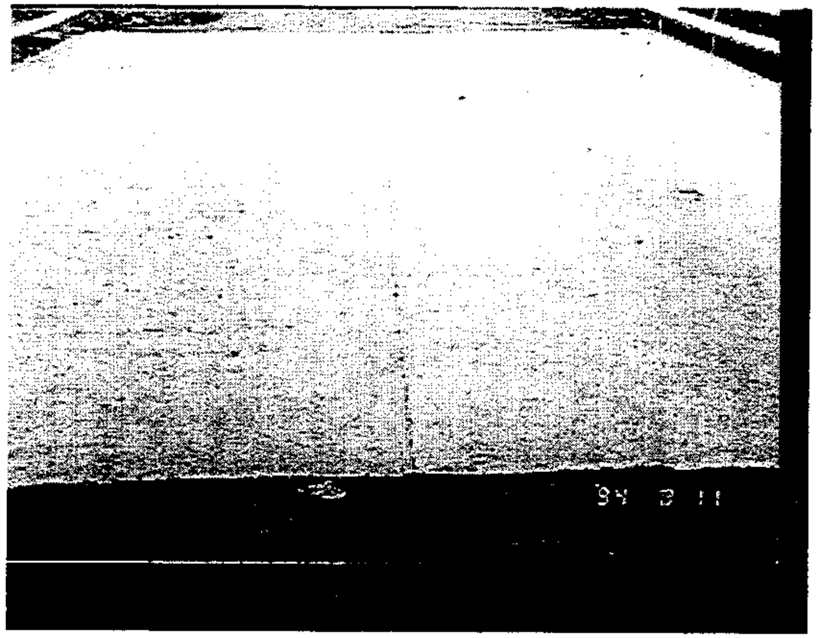


Figure 2.29 Appearance of isotropic deck at Susquehanna

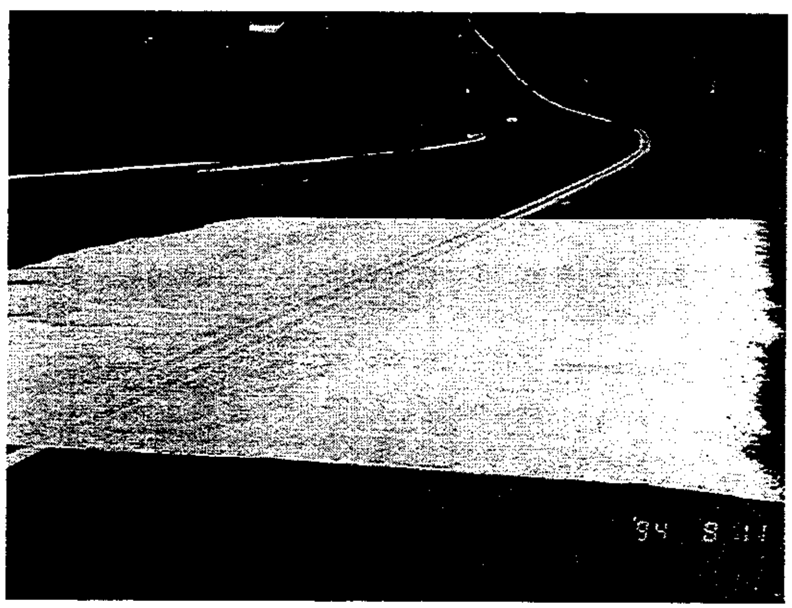


Figure 2.30 Appearance of AASHTO deck along SR 0371

Table 2.7 District 4-0 bridge deck summary

County	Susquehanna	Wyoming
District	4-0	4-0
SR Number	11	11

Number of Spans		1		4	
Span Lengths		75'-6",		100'-0 3", 140'-0", 140'-0", and 100'-0 3".	
Width		43'-6"		41'6"	
Type of Deck		Isotropic		AASHTO	
Skew (degrees)		90		90	
Girder spacing		8' 8.5"			
Completion of Construction		N/A		1990	
Material Cost/ And Labor cost		$2.67ft^2$ (ositive reinforcement)			
Total Crack Length	**Transverse**	Year 1	N/A	Year 1	N/A
		Year 3	0	Year 3	N/A
		Year 5	N/A	Year 5	N/A
	Longitudinal	Year 1	N/A	Year 1	N/A
		Year 3	*75.5'	Year 3	N/A
		Year 5	N/A	Year 5	N/A
	Diagonal	Year 1	N/A	Year 1	N/A
		Year 3	N/A	Year 3	N/A
		Year 5	N/A	Year 5	N/A
Number of Permit Loads		N/A		N/A	
Miscellaneous		Structure #: S-18165A Bridge type: Spread box beam Girder size: 48"x45" Area: 365 sq. yds		Structure #: S-17724 Girder size: 27"x48" Bridge Type: P/S concrete spread box beam Area: 2115 sq. yds.	

* Appears to generate from construction joint N/A – Not available

Table 2.8 District 4-0 bridge deck summary

County	Wayne	Wayne
District	4-0	4-0
SR Number	0371	0170
Number of Spans	1	

Span Lengths		41' 6"		40'-0"	
Width		28' curb to curb		31'-6" (out-to-out	
Type of Deck		Isotropic		AASHTO	
Skew (degrees)		90		90	
Girder spacing		7' 10"		7'-10"	
Completion of Construction		1990		N/A	
Material Cost/ And Labor cost		$\$2.67\text{ft}^2$ (positive reinforcement)			
Total Crack Length	**Transverse**	Year 1	N/A	Year 1	N/A
		Year 3	0	Year 3	N/A
		Year 5	N/A	Year 5	N/A
	Longitudinal	Year 1	N/A	Year 1	N/A
		Year 3	0	Year 3	N/A
		Year 5	N/A	Year 5	N/A
	Diagonal	Year 1	N/A	Year 1	N/A
		Year 3	0	Year 3	N/A
		Year 5	N/A	Year 5	N/A
Number of Permit Loads		None		N/A	
Miscellaneous		Structure #: S-18195 Bridge type: Spread box beam Girder size: 48 in.x 21 in. Area: 129 sq. yds		Structure #: S-18057 Girder size: 48 in.x 21 in. Area: 128 sq. yds	

2.6 District 5-0

The isotropic bridge in this district is located at Scarlet Mills Bridge, Berks County, on SR 2031 (Figure 2.33) and was constructed in 1993. The bridge is a simple span, prestressed concrete box beam system with a span length of 70 feet center to center and a width of 31 feet-6 inches out to out. The deck is supported on four 48-inch x 45-inch box beams, spaced 8 feet center to center and skewed 80°. The slab thickness ranged from 8.5

inches to 10.25 inches The reinforcement consists of two mats of #4 bars placed 12 inches in the longitudinal direction and 11.5 inches center to center in the transverse direction. The longitudinal bars were placed parallel to the girders, while the transverse bars were place in the direction of the skew. Additional bars were place in the ends and overhangs in accordance with the Standard Details. Some general properties of the deck are summarized in Table 2.9.

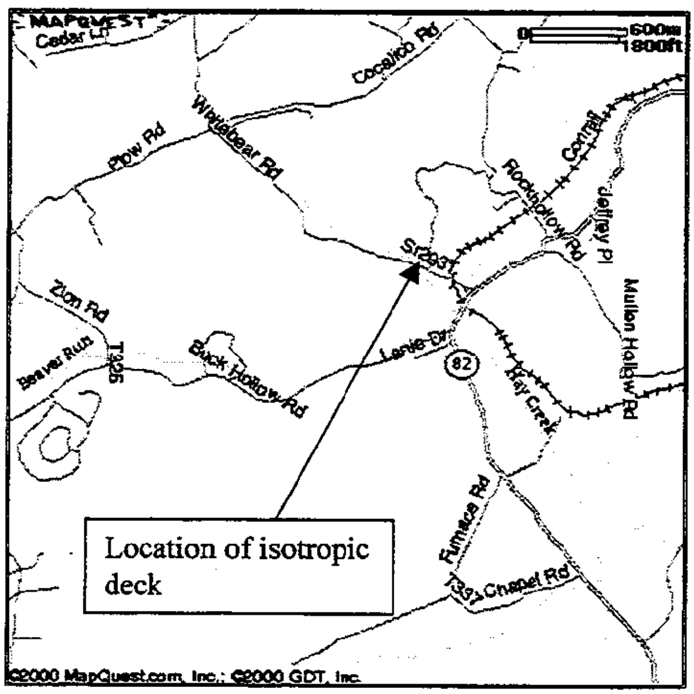


Figure 2.33 Location of Isotropic deck in Berks County

Table 2.9 Summary of bridge deck information for District 5-0

County	Berks	Berks
District	5-0	5-0
SR Number	82	2031
Number of Spans	1	1
Span Lengths	N/A	68'-5.75"
Width	N/A	31'-6" out-to-out

Type of Deck		AASHTO		Isotropic	
Skew (degrees)		N/A		80°	
Girder spacing		N/A		8'-0"	
Completion of Construction		N/A		1993	
Material Cost/ And Labor cost		N/A			
Total Crack Length	**Transverse**	Year 1	N/A	Year 1	8
		Year 3	N/A	Year 3	N/A
		Year 5	N/A	Year 5	N/A
	Longitudinal	Year 1	N/A	Year 1	84
		Year 3	N/A	Year 3	N/A
		Year 5	N/A	Year 5	
	Diagonal	Year 1	N/A	Year 1	13
		Year 3	N/A	Year 3	N/A
		Year 5	N/A	Year 5	N/A
Number of Permit Loads		None recorded		None recorded	
Miscellaneous				Structure#: 518193 Area: 245 sq. yds Girder size: 48 in.x 45 in. Area: 241 sq. yds.	

The calculated concrete cracking moment was determined to be 5,056-feet-lb/ft, while the AASHTO design moment was 3,468-foot-lb/ft, and the Newmark's (1949) moment 3,871-foot-lb/ft. Since the design moment is less than the cracking strength of the deck, cracking due to service load is not expected.

No construction photos were available; however, there were photos taken during the routine field inspection that was conducted in August 1994 (Figure 2.34) and June 1996 (Figure 2.35). From the photograph, Figure 2.35, two longitudinal cracks ran the entire length of the bridge in the central region of the deck; however, the field reports stated that the cracks were minor. A sketch from the prior field visit showed that there was only one longitudinal crack along the entire length of the deck, part of which is seen in Figure 2.34.

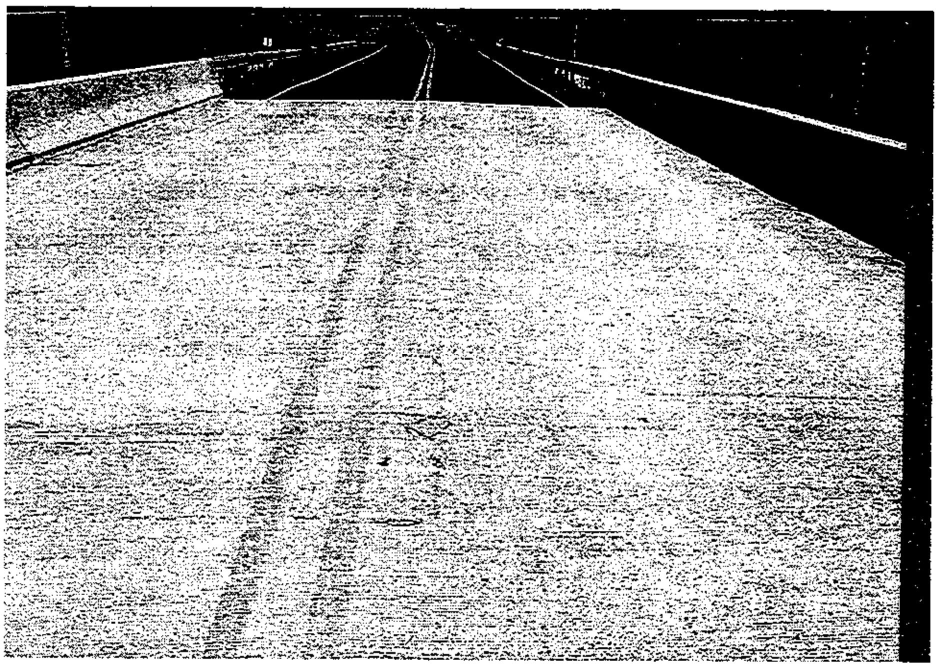


Figure 2.34 SR 2031 in August 1994 (1 year after construction)

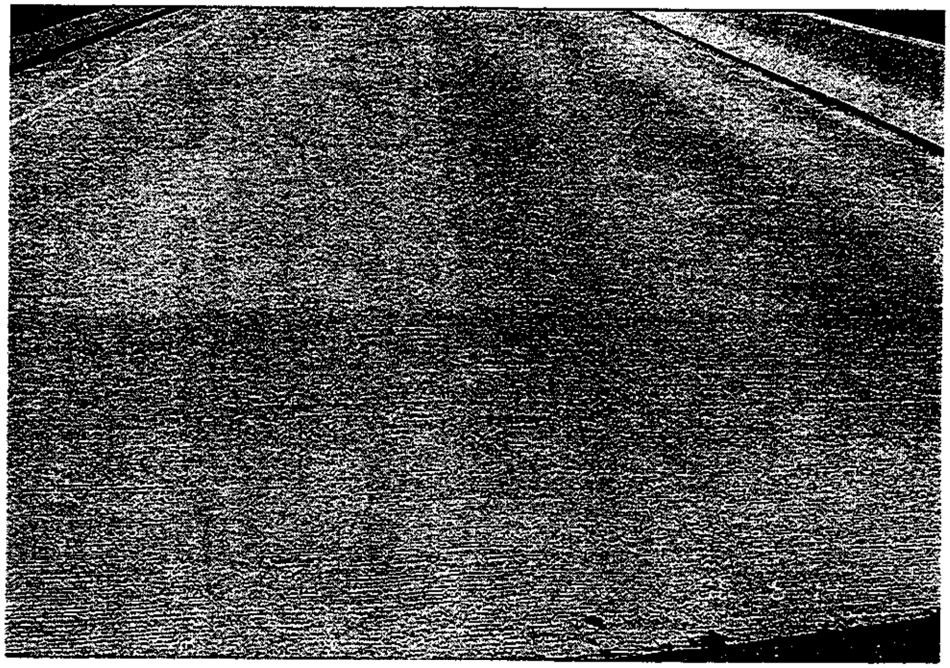


Figure 2.35 Bridge on SR 2031 June 1996 (3 years after construction)

2.7 District 9-0

Two bridges were investigated in this district. The isotropic deck, constructed in 1993, was located over Bald Eagle Creek on SR 4027 in Blair County (Figure 2.36), and its AASHTO control counterpart, constructed in 1992, was located on SR 403 over Stony Creek River, Quemahoming Township, Somerset County (Figure 2.37). Both decks were located in rural areas and appear to be experiencing low traffic volume. Figures 2.38 to 2.39 are photos of the decks taken during routine field visits.

The former is a two-span continuous concrete spread box beam bridge, with each span 66 feet-1 ½ inches, resulting in a total length of 132 feet-3 inches It had a total width of 38 feet-6 inches out to out, and a curb-to-curb width of 35 feet that supported two lanes of traffic.

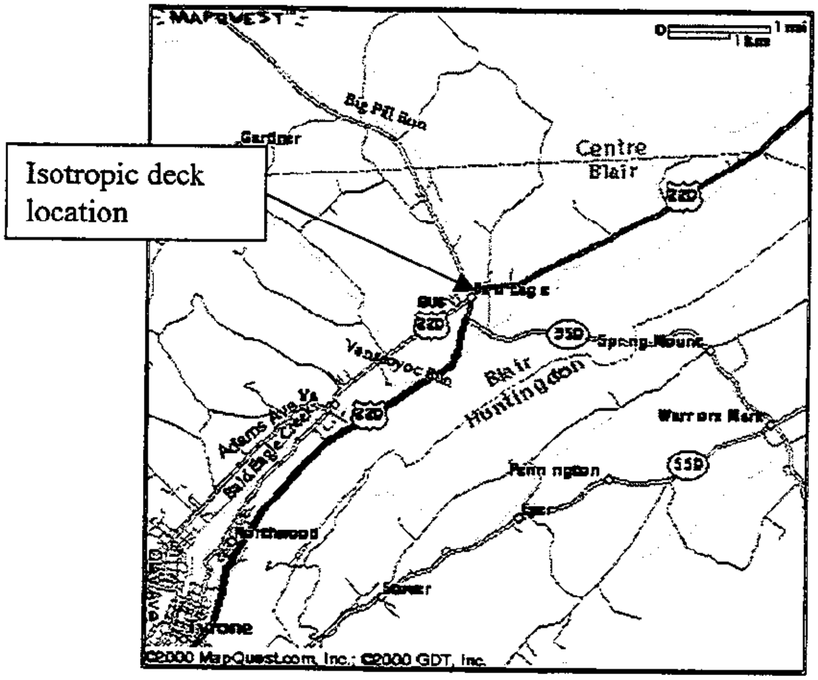


Figure 2.36   Location of structure in Blair County along SR 4027

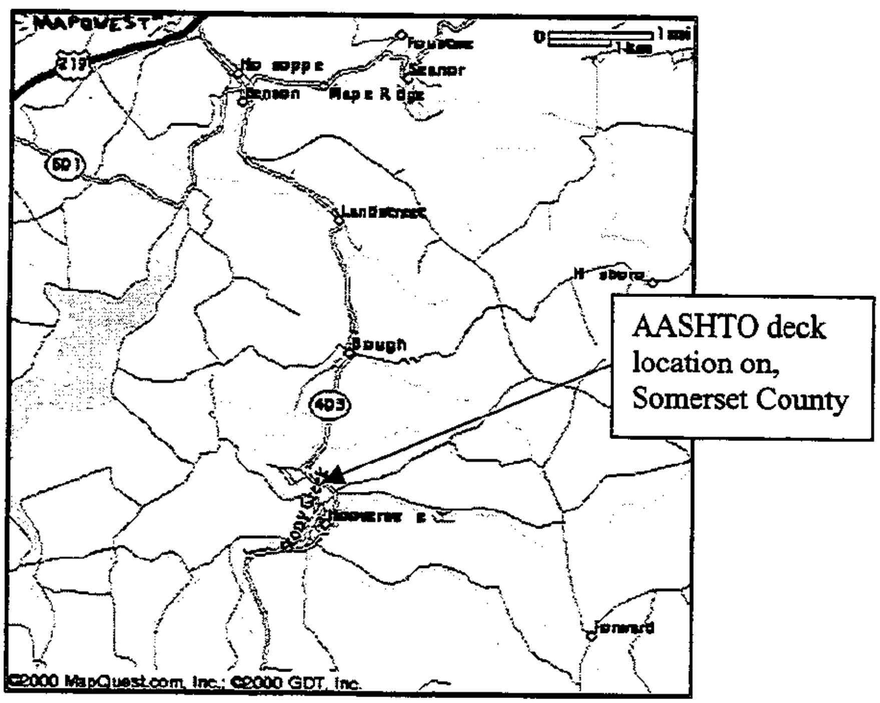
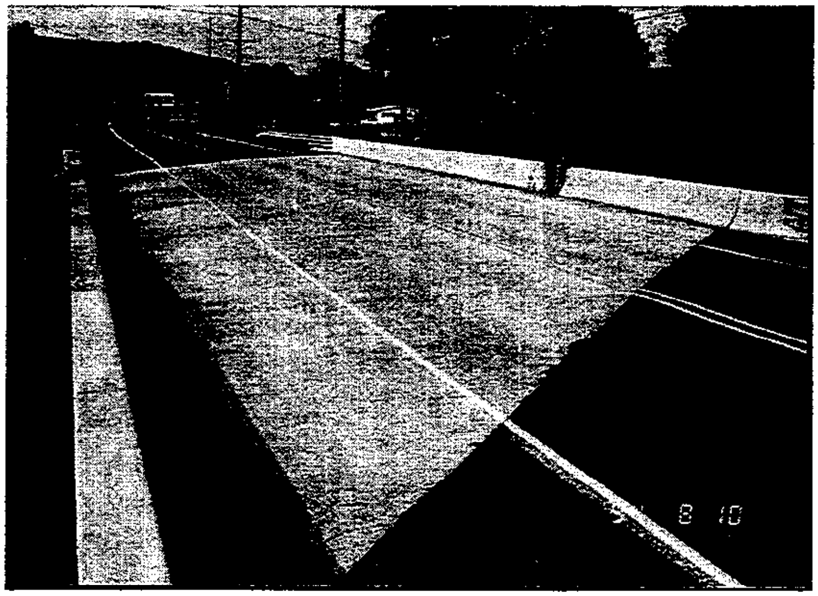
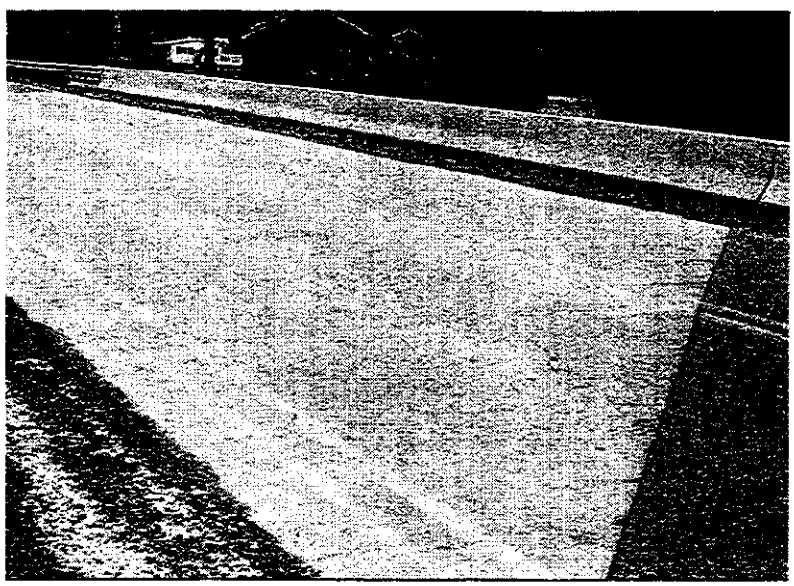


Figure 2.37   Location of AASHTO bridge deck along SR 40

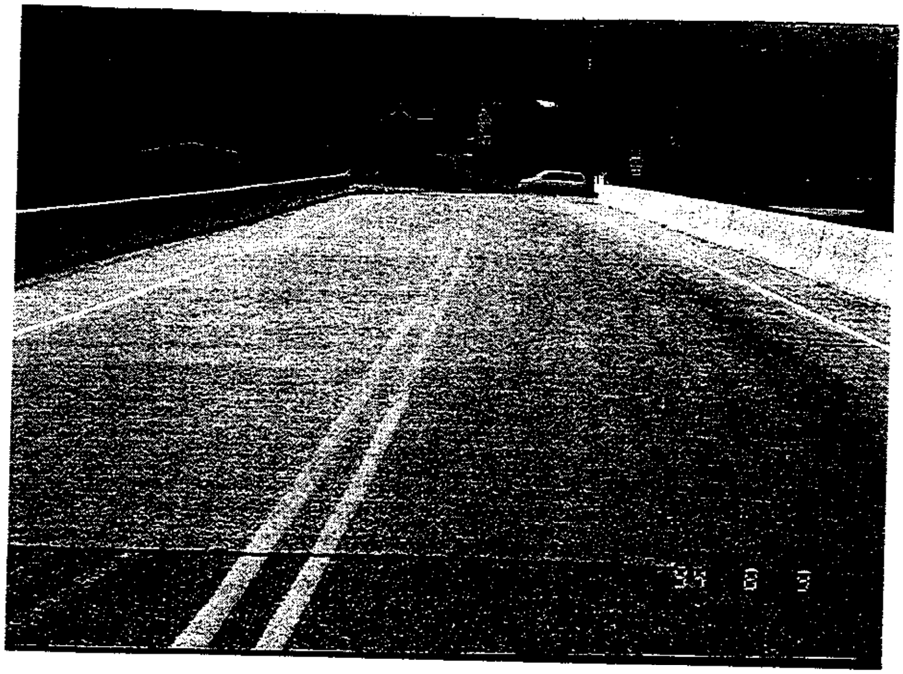


(a)

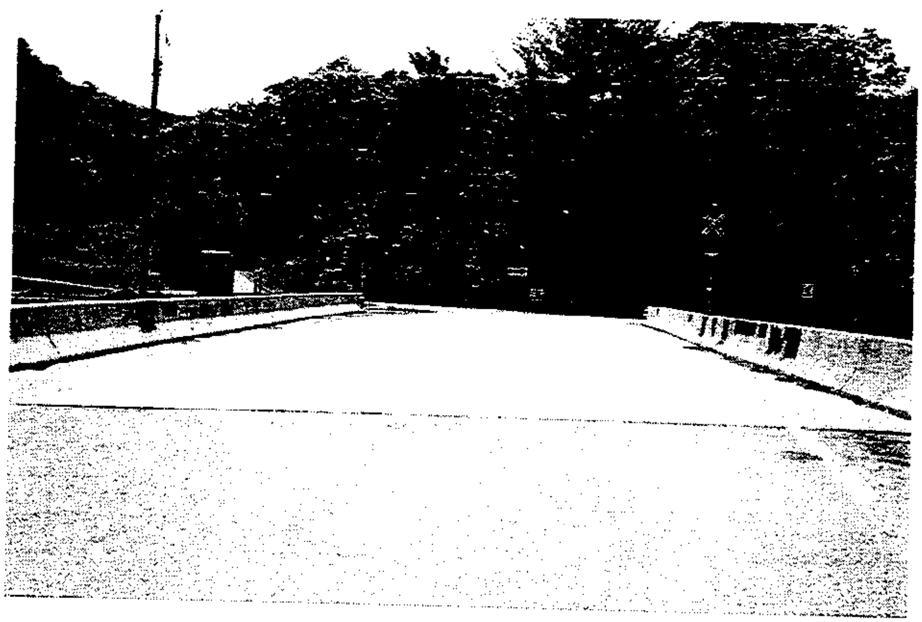


(b)

Figure 2.38 Isotropic bridge on SR 4027, (a) August 1994, (b) June 2000



(a)



(b)

Figure 2.39 Bridge location on SR 403, (a) August 1994, (b) June 2000

Supporting the deck are four 48-inch x 36-inch concrete box beams spaced 10 feet-2 inches on center with a 90° skew. A summary of the general properties is shown in Table 2.10.

Table 2.10 Summary for District 9-0

		Somerset		Blair	
County		Somerset		Blair	
District		9-0		9-0	
SR Number		403		4027	
Number of Spans		2		2	
Span Lengths		2 @ 66' 1 ½"		2 @ 44'-10 $^{13}/_{16}$"	
Width		35'-0" curb-to-curb 38'-6" out-to-out		40'-0" curb-to-curb 43'-6" out-to-out	
Type of Deck		AASHTO		Isotropic	
Skew (degrees)		90		45	
Girder spacing		4 @ 10'-2"		5 @ 9'-0"	
Completion of Construction		1992		(Plan recommended 10-9-92)	
Material Cost/ And Labor cost					
Total Crack Length (ft)	**Transverse**	Year 1	12	Year 1	5
		Year 3	N/A	Year 3	N/A
		Year 5	N/A	Year 5	10
	Longitudinal	Year 1	20	Year 1	5
		Year 3	N/A	Year 3	N/A
		Year 5	N/A	Year 5	40
	Diagonal	Year 1	0	Year 1	11
		Year 3	N/A	Year 3	N/A
		Year 5	N/A	Year 5	15
Number of Permit Loads		N/A		N/A	

Miscellaneous	Structure #: S-18251 Bridge type: Spread box beam Girder size: 48 in.x 36 in. Minor cracking were observed Area: 497 sq. yds	Structure#: S-18611 Bridge type: 2 span continuous composite prestress concrete spread box beam bridge Girders size: 48 in. x 27 in. Area: 399 sq. yds

The isotropic deck was also a two-span continuous concrete spread box beam bridge with each span having equal lengths, 44 feet-10 $^{13}/_{16}$ inches It was 8.5 inches thick and was supported on five 48-inches x 27-inch girders spaced at a clear span of 5 feet-9 inches. Table 2.10 summarizes some general properties of the deck, including some field data.

A field visit in August 1994 revealed minor cracking on the isotropic deck. Both the isotropic and AASHTO decks were photographed, and the crack locations were sketched on the schematic diagrams. In June 2000 the AASTO and isotropic deck were visited for a follow-up field inspection. The two decks appeared to be in fairly good condition. The main difference was that the AASTO had a higher crack density, and it suffered more extensive longitudinal cracking than its isotropic counterpart. Four longitudinal cracks observed on the AASHTO deck were approximately over the edges of the two central box girders. Some of the cracks in the central regions of the isotropic deck were covered with asphalt that made it impossible to see their severity; however, from the lengths covered, the cracks were only a few feet long.

2.8 District 10-0

Two bridges for this investigation were constructed in District 10-0. Both the isotropic deck and its AASHTO control deck were located along SR 422 (Figure 2.40). The isotropic deck was located over Buffalo and Pittsburgh Rail Road and Bonnie Brook Creek. Each bridge carried two lanes of traffic. In 1990 the eastbound directions of both bridges were constructed, while the westbound directions of the bridges were

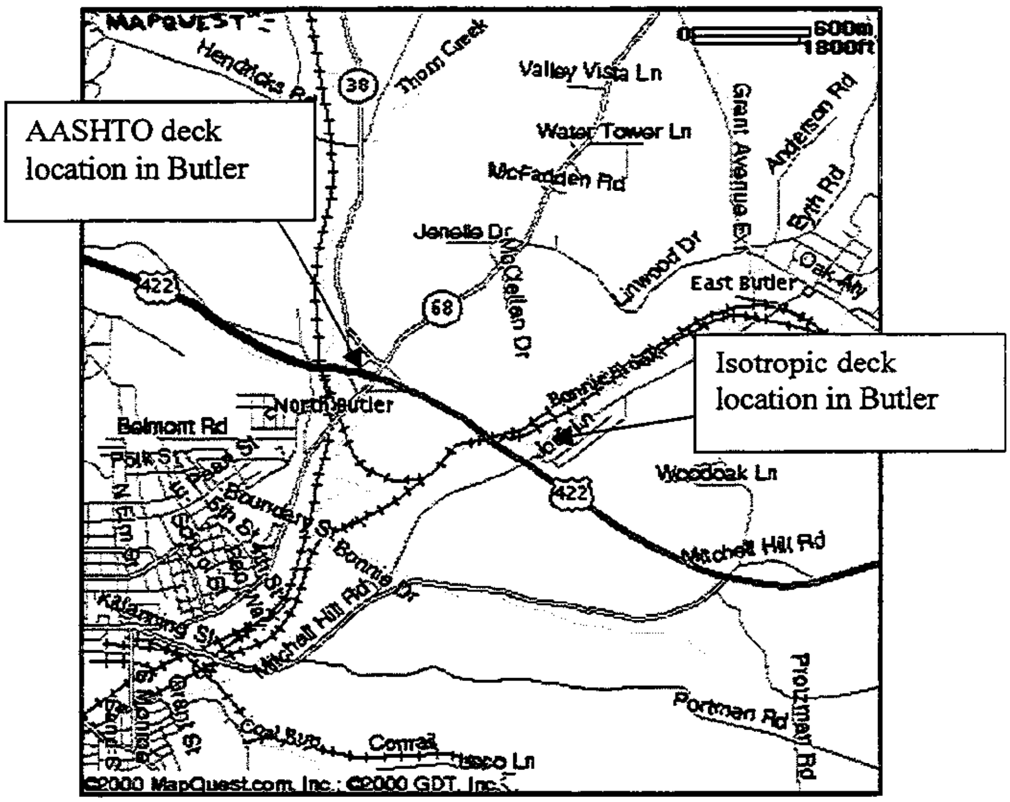


Figure 2.40 Location map for bridges on SR 422

constructed the following year. The AASHTO decks were placed on a four-span composite steel I-beam girders, with a total length of 487 feet. The individual spans ranged from 107.5 feet to 149.0 feet. Each deck had a width of 32.75 feet. The bridge had a total width of 65.5 feet. Each deck was supported on four w15x80 steel I-beams on a 60-degree skew. The beam spacing on span 1 was 9.0 feet, but was variable along span 4. The deck was 8.5 inches thick for spans 1-3 and 8 inches for span 4, which was the eastern most span length of 104.5 feet. Photographs of the decks are shown in figures 2.41 to 2.43. Typical reinforcement in the positive moment region of the deck consists of #4 bars spaced 12 inches center to center longitudinally in the top mat, and #5 bars were placed 8.5 inches center to center in the bottom mat. A typical negative region consists of #5 bars and #6 bars placed 12 inches on

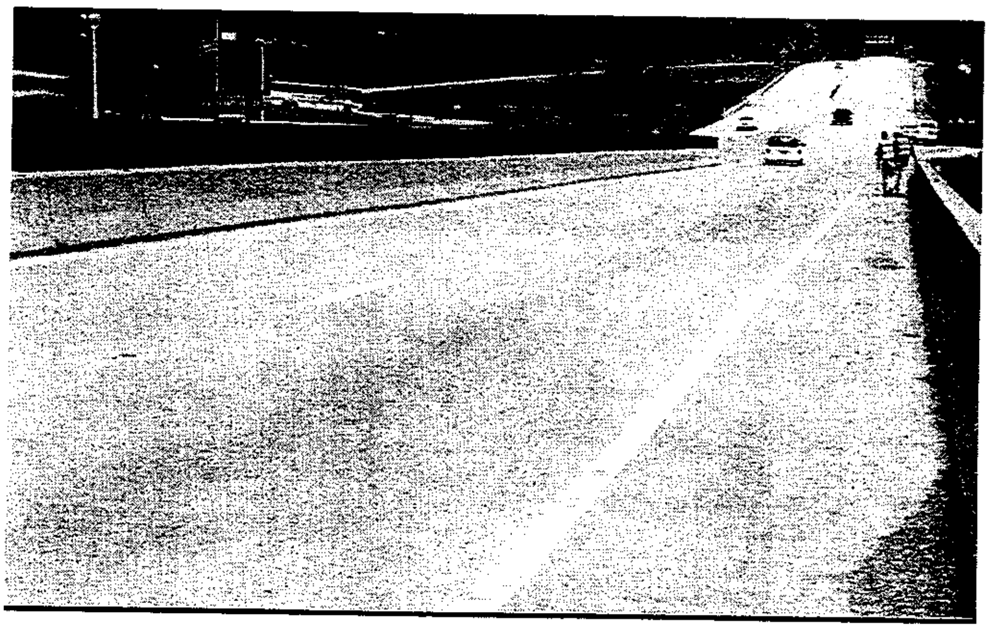


Figure 2.41 Eastbound section of bridge along SR 422 June, 2000 (isotropic)

Figure 2.42 Westbound section of bridge along SR 422, June 2000 (Isotropic)

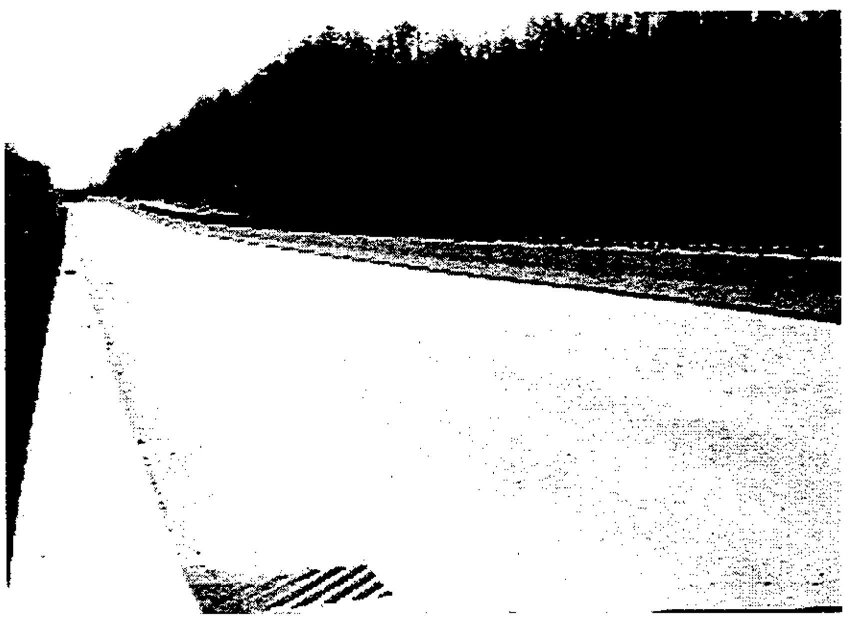


Figure 2.43 AASHTO and Isotropic Bridge Deck in 2001(Eastbound)

centers in the longitudinal direction between the top mat. However, the bottom longitudinal and transverse reinforcement pattern was identical to the positive moment regions. Number 6 bars were also placed 7.5 inches on centers in the transverse direction in both positive and negative moment regions.

The isotropic deck was placed on a six-span composite I-beam bridge with a total span length of 472 feet. The spans on either side was 56 feet and 70 feet, while the four central spans were 86.5 feet each, resulting in a total length of 472 feet. It had a total out-to-out width of 65.5 feet, with the eastbound and westbound lanes separated by a three-foot slab in both directions resting upon six W36x160 steel I-beams on a 48°30.5' skew. The beams were spaced 5 feet-7inches between the beams.

Reinforcement consists of #4 bars spaced 12 inches in positive moment regions. The placement of the reinforcement was either perpendicular or parallel to the girders. In the negative moment regions #6 bars were placed in the top mat between the #4 bars and were spaced 12 inches on centers in the longitudinal direction of the bottom mat.

Table 2.11 Summary of bridge deck information for District 10-0

County	Butler	Butler
District	10-0	10-0
SR Number	SR 422	SR 422
Number of Spans	4	6
Span Lengths	115/149/115/107	56, 4 at 86' each, 70
Width (ft)	2x32.75 = (65.5)	2x32.75 = (65.5)
Type of Deck	AASHTO	Isotropic
Skew (degrees)	60	48°33'
Girder spacing	5'-7" center-to-center	9' center-to-center
Completion of Construction	1990 (eastbound) 1991 (westbound)	1990 (eastbound) 1991 (westbound)
Material Cost/ And Labor cost		

Total Crack Length	**Transverse**	Year 1	N/A	Year 1	N/A
		Year 3	49	Year 3	36
		Year 5	N/A	Year 5	N/A
	Longitudinal	Year 1	N/A	Year 1	N/A
		Year 3	0	Year 3	0
		Year 5	N/A	Year 5	N/A
	Diagonal	Year 1	N/A	Year 1	N/A
		Year 3	13	Year 3	0
		Year 5	N/A	Year 5	N/A
Number of Permit Loads		N/A		N/A	
Miscellaneous		Structure#: 17983 Thickness: 7.5" minimum Area: 3246 sq. yds Type: 4-span composite steel multi-girder bridge		Structure#: 17824 Deck Thickness: 8.5 in. Area: 3146 sq. yds Type: 6-span composite multi-girder bridge	

A field inspection carried out in August 1994 indicated that there were some minor cracks. Their locations were sketched on a schematic diagram of the bridge, and the decks were photographed.

The follow-up field inspection in June 2000 also revealed that both decks have been performing very well, and there was low incidence of cracks in both decks. The most severe cracks were located approximately in the same regions of both the eastbound and westbound decks. These cracks were transverse on both the isotropic decks; however, the crack width on the westbound appeared slightly wider. This observation was visual; the crack width was not measured, being relatively minor. At the time of visit, both bridges were moderately traveled by heavy trucks, as well as lighter vehicles. The AASHTO deck was also in very good condition, having only a few minor transverse cracks.

2.9 Summary:

In general, both the isotropic and AASHTO bridge decks appeared to be performing well. In all the cases investigated in the field, only minor cracks were observed, and the crack density was low. A general summary of the decks is shown in Table 2.12.

Missing information includes cost data for most of the bridges. The only bridge deck on SR 62 in District 1-0 in Venango County for which detailed cost was available resulted in an estimated material savings of $35,000.00 over the conventional AASHTO deck (Savings of $1.00/sq. ft of the deck). Costs of the other bridge decks were estimated with some assumptions. Because isotropic decks utilizes less steel than their AASHTO counterparts, it is expected that labor and material cost may be lower in the construction of these decks compared with their AASHTO counterparts; however, no direct cost was obtained from the bridge decks under investigation. An estimate of the construction cost of these decks was obtained by categorizing the cost into some specified units based on some quantity of epoxy coated reinforcement, labor cost, and

Table 2.12 Bridge deck summary data

SR #	Date Constructed	Type of Deck	County	Engineer-ing District	Number of Spans	Date of Inspec-tion	Total crack Length (ft)		
							T	L	D
062	1991	Iso.	Vernango	1-0	5	12/91 8/94 6/00	63 96 128	0 104 156	0 0 0
045	1991	Iso.	Union	3-0	1	8/94 6/00	0 0	0 0	0 0

973	1990	AA.	Lycoming	3-0	1	8/94 6/00	0 0	0 2	0 3
011	N/A	Iso.	Susquehenna	4-0	1	8/94	0	*75.5	0
371	1990	Iso.	Wayne	4-0	1	8/94	N/a	N/a	N/a
2031	1992	Iso.	Berks	5-0	1	8/94	10	84	11
082	N/A	AA.	Berks	5-0	1	N/a	N/a	N/a	N/a
403	1992	AA.	Somerset	9-0	2	8/94 6/00	55 586	29 259	9 8
4027	1993	Iso.	Blair	9-0	2	8/94 5//98 6/00	5 10 23	5 40 63	11 15 23
422	1990	Iso.	Butler	10-0	6	8/94 6/00	49 252	0 0	0 0
422	1991	AA.	Butler	10-0	4	8/94	55	0	17
170	N/A	AA.	Wayne	4-0	N/A	N/A	N/A	N/A	N/A
011	1990	AA.	Wyoming	4-0	4	N/a	N/A	N/A	N/A

AA - AASHTO type deck N/A – not available

Iso. – Isotropic deck * Appears to generate from construction joint

material cost. One unit of epoxy-coated reinforcement was taken as one pound and was given an estimated cost of $0.85. The labor unit was defined as the cost required to place one linear foot of reinforcement on the deck. Some of these data are summarized in Table 2.13 below.

Table 2.13 Bridge Deck Material and Labor Cost (Sheftick 1992)

Bridge Deck	Lb. Of Reinforcement/sq. ft	Material Cost: ($)/sq. ft	Labor Units/ sq. ft
Isotropic	2.67	2.27	4.0
SR 422	6.95	5.90	5.61
SR 973	3.43	2.92	3.92

(*Comparison made for positive moment region only)

Reinforcement cost for the SR 45, Union County, isotropic bridge deck was 78 percent of the reinforcement cost of the deck located on SR 973, Lycoming County, AASTHO deck; however, labor units costs were nearly equal for these two decks. The reinforcement cost for the Butler County isotropic deck was significantly lower than its AASHTO counterpart; it cost 38.5 percent of its AASHTO counterpart. In addition to that, the labor unit cost of the isotropic deck slab was 71 percent of its AASHTO counterpart.

Bottom deck cracking would not be expected in any PENNDOT isotropic deck constructed to date. The only decks under investigation where bottom cracking is expected is the AASHTO deck on SR 422 since its cracking strength was slightly lower that the service load moment. However, any observation of bottom cracking could not be observed since the decks had cast-in-place forms. Some of the surface cracking are shown in the appendix at the end of this document.

It is premature to draw any conclusion between the performance of isotropic and AASHTO-type deck; however, as far as the field data revealed, the isotropic decks have performed satisfactorily. In the isotropic deck at Union County, there were absolutely no cracks observed and there had been no record of spalling in any of the decks.

Permit loads were recorded only on one of the bridges. It was found that it would be difficult to keep track of the permit loads, so it was possible that some of these loads could have been missed. The permit load was recorded in District 3-0 along SR 45, which was an isotropic deck; however, a visit to that bridge revealed no visible cracks.

CHAPTER 3. METHODS OF ANALYSIS

This chapter gives details of the different concepts and procedures employed in the design of conventional AASHTO and isotropic decks. It starts with an explanation of the traditional ASSHTO design method in the design of bridge decks in the following sections. Details of the concepts and procedure in the empirical design method used to design isotropic bridge decks are also given. Finally, a comparison is made between the empirical method and the standard AASHTO design method.

3.1 AASHTO Bridge Deck Design Procedure

The standard AASHTO design procedure uses approximate elastic methods to analyze and design conventional bridge decks. Once the dimensions of the bridge are established, and the girder spacing is determined, the deck slab can be designed. The first step involves the determination of the average deck thickness. The loads on the slab, including dead and live loads, are determined, after which the loads are used in the analysis of moment in both the continuous section and the overhang. The area of the required steel is then determined based on the factored service load moments. Appropriate bars and spacing are then chosen. The moment capacity and steel stress are checked against some specified conditions. Temperature and shrinkage reinforcement are then calculated in addition to those required for moment. In some cases, shear and bond may be checked, but these checks are not necessary for the AASHTO decks. Finally, the deck is analyzed for serviceability, which includes fatigue stress limit, crack control, and deflection.

The following sections give details of the various steps described above.

3.1.1 Deck thickness

The minimum deck thickness is set at 7 inches; however, depending on the dimensions of the deck, the minimum deck thickness may be determined from the following relationships.

For slabs supported on box girders, the minimum thickness is obtained by multiplying the span length by 0.06 for simple spans and 0.055 for continuous spans. In simple slabs where the main reinforcement is parallel to traffic, the following formula is used to determine the minimum slab thickness:

$$t_{min} = 1.2(S+10)/30 \qquad (3.1)$$

For continuous decks with main reinforcement parallel to traffic:

$$t_{min} = (S+10)/30 \geq 0.542 \qquad (3.2)$$

Where:

S = span length

3.1.2 Span lengths (AASHTO 3.24.1.2)

For simple span, the span length is the distance from center to center of the supports but should not exceed clear span plus thickness of slab. The effective span length for slabs continuous over two or more supports is the clear span in slabs monolithic with beams or walls (without haunches). For slabs supported on steel stringers, however, the span length (S) is the distance between edges of flanges plus one half of the stringer flange width.

3.1.3 Loading:

Bridges supporting Interstate highways or other highways which carry, or which may carry, heavy traffic are designed for HS20-44 loading or an Alternate Military loading of two axles four feet apart with each axle weighing 24, 000 pounds, whichever produces the greatest stress. The standard HS20-44 truck is shown in Figure 3.1.

The HS loading consists of a tractor truck with semi-trailer or the corresponding lane load. The HS loading is designated by the letters HS followed by a number indicating the gross weight in tons of the tractor truck (AASHTO 3.5.5). Highway live loads are increased for the slabs to account for dynamic, vibratory, and impact effects (AASHTO 3.8.1). The amount of impact allowance or increment is expressed as a fraction of the live load stress and is determined by the formula:

$$I = 50/(L+125) \tag{3.3}$$

where:

I = impact fraction (maximum 30 %)

L = length in feet of the portion of the span that is loaded to produce the maximum stress in the member (AASHTO 3.8.2).

In designing slabs, the centerline of the wheel is placed one foot from the face of the curb. If the curbs or sidewalks are not used, then wheel load is placed 1 foot from the face of the rail.

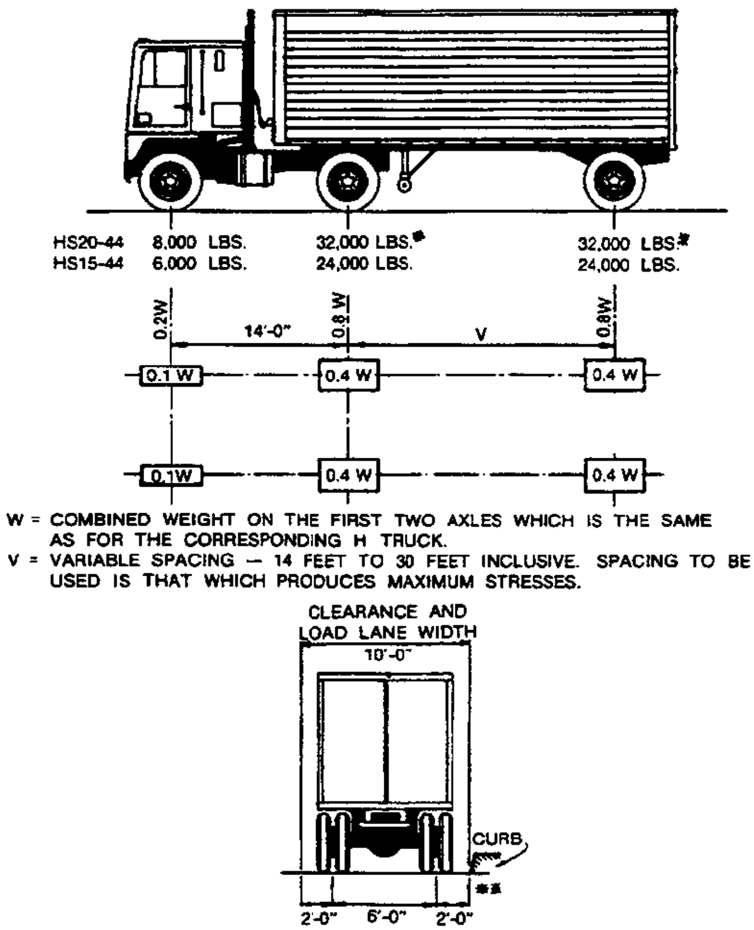


Figure 3.1 AASHTO standard HS truck

(AASHTO 3.24.2.1). In service load design, the combined dead live, and impact stresses for this loading should not be greater than 150% of the allowable stresses. In load factor design, 1.0 may be used as the beta factor in place of 1.67 for the design of deck slabs. Wheel loads are not applied to sidewalks protected by a traffic barrier (AASHTO 3.24.2.2).

Group I loading (for load factor design) = $\gamma[\beta_D.W_D+\beta_L(W_{L+I})]$ (3.4)

Where:

W_D = dead load

W_{L+I} = live load plus impact

3.1.4 Bending Moment

The bending moment per foot of slab is calculated according to methods given below, unless more exact methods are used, considering the tire contact area. The tire contact area is needed for exact methods; however, a treatment of the exact method is not given in this document.

In this section:

S = effective span length, in feet, as defined in the previous section;

E = width of slab in feet over which a wheel load is distributed;

P = load on one rear wheel of truck (P15 or P20);

P15 = 12,000 pounds for H 15 loading;

P20 = 16,000 pounds for H 20 loading (AASHTO 3.24.3).

To determine the live load moment in the slab parallel to traffic (spans 2 to 24 feet inclusive), the following formula is used:

$$M_L = \left(\frac{S+2}{32}\right)P \quad \text{(foot-pounds per foot-width of slab)} \qquad (3.5)$$

where:

M_L = live load moment in foot-pound per foot width of slab

S = span length in feet

P = live load in pounds

For example, the live load moment for HS 20 loading would be as follows:

$$M_L = \left(\frac{S+2}{32}\right)16{,}000 \quad \text{(ft-lb/ft)}$$

Similarly for HS 15 loading:

$$M = \left(\frac{S+2}{32}\right)12{,}000 \quad \text{(ft-lb/ft)} \tag{3.6}$$

In slabs continuous over three or more supports, a continuity factor of 0.8 is applied to the above formulas for both positive and negative moment.

A number of methods can be use to determine the moment perpendicular to traffic; however, for simple spans, the maximum live load moment per foot width of slab, without impact, is closely approximated by the following formulas:

Live load moment (LLM) = 900S foot-pounds (for HS 20 loading, Spans up and including 50 feet)

LLM = 1000S (1.30S -20) foot-pounds (for HS 20 loading, Spans 50 feet to 100 feet)

The above formula can also be used to determine the live load moment by other standard trucks. For example, to determine the moment due to the HS 15 loading, use 75 percent of the value obtained for the HS 20 loading.

Moments in continuous spans are determined by suitable analysis using truck or appropriate loading.

Using the following formulas for distribution of loads on cantilever slabs, the slab is designed to support the load independently of the effects of any edge support along the end of the cantilever.

Determination of Moment Parallel to Traffic in Cantilever Section:

The moment per foot of slab is determined using the relation $M = (P/E)\ X$ foot-pounds, in which X is the distance in feet from load to point of support.

E is the wheel load distribution on elements perpendicular to traffic, and it is determined from the formula:

$E=0.8X + 3.75$

The slabs are designed to support the load independently of the effects of any edge support along the end of the cantilever.

This moment is added to the dead load moment in order to compute the total moment.

The dead load moment is computed from the following formula:

$$M_D=W_DL^2/2+W_{C+P}L \qquad (3.7)$$

where:

M_D=Dead load moment

L= distance of center of gravity of parapet from main end support

W_D = dead load

S = span length

W_{C+P}=weight of parapet and curb

The moment perpendicular to traffic per foot width of slab is calculated similar to the preceding section using the formula:

$M = (P/E)X$ foot-pounds.

Where X is the distance in feet from the load to the point of support.

The distribution width for each wheel load on the element parallel to the traffic is determined as follows:

$E=0.35X+3.2$, but does not exceed 7.0 ft (3.8)

3.1.5 Shear and Bond:

Slab design for bending moments in accordance with the standard AASHTO procedure is considered satisfactory in bond and shear; therefore, no analysis is required in this section.

3.1.6 Determination of Amount of Reinforcement:

The minimum amount of reinforcement provided should be able to develop an unfactored moment of at least 1.2 times the cracking moment ($1.2M_{cr}$) unless the area of reinforcement provided is at least one-third greater than that required by analysis.

Equations 3.9 to 3.13 are use to determine the amount of reinforcement in the positive region of the slab. In this example, reinforcement in the compressive face of the deck slab will not be considered.

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right) \quad (3.9)$$

where:

$$a = A_s f_y / (0.85 f_c' c b) \quad (3.10)$$

$d = t$ – wearing surface – bottom cover – ½ diameter of bar

t = thickness of slab

A_s = steel area

b = width of section

f_y = steel strength

ϕ = reduction factor

M_n = service load moment

Since A_s is the only unknown, it can be determined from the equations 3-9 and 3-10. The actual value of **a** is then determined after the appropriate bars are chosen. The value of **a** is then used to determine the moment capacity (M_u) of the slab. The area of reinforcement for the different regions of the deck is determined by substituting the appropriate moment value in the formula above and solving for A_s.

The maximum steel ratio (ρ_{max}) is obtained from the following formula:

$$\rho_b = \frac{0.85\beta f'_c c}{f_y}\left(\frac{87000}{87000 + f_y}\right) \tag{3.11}$$

where f'_c and f_y are in pounds per square inch.

$$\rho_{max} = 0.75\rho_b \tag{3.12}$$

After the steel reinforcement is selected, the formula below is used to determine the ultimate moment capacity of the designed slab.

$$M_u = \phi A_s f_y (d - a/2) \tag{3.13}$$

3.1.7 Checking minimum steel

The AASHTO code specifies another means of checking minimum steel based on the cracking moment and unfactored service load moment. If the moment capacity (M_u) is equal or larger than $1.2M_{cr}$, then the slab is sufficiently reinforced. The cracking moment is determined from the following formulae:

$$M_{cr} = f_r I / y_t \tag{3.14}$$

$$f_r = 7.5\sqrt{f'c} \tag{3.15}$$

$$E_c = 33\, w^{1.5}\sqrt{f'c} \tag{3.16}$$

$$E_s = 29{,}000{,}000$$

Where E_c and E_s are the elasticity modulus for concrete and steel, respectively.

$$n = E_s/E_c \tag{3.17}$$

n = moduli ratio

$$y_t = [bh(h/2)+(n-1)A_s d_c + \tfrac{1}{2} D_{bar})]/(bh + (n-1)A_s) \tag{3.18}$$

$$I_{cg} = (1/12)\, bh^3 + bh(d')^2 + (n-1)As(d'')^2 \tag{3.19}$$

Where d' and d" are the distances of the center of the concrete and reinforcement from the center of gravity of the reinforced section, respectively.

3.1.8 Distribution Reinforcement: (AASHTO 3.24.10)

Additional reinforcement is added for the lateral distribution of concentrated live loads. The reinforcement should be placed transversely to the main reinforcement in the bottom of the slab in the positive moment region of the deck slab. The amount of distribution reinforcement is a percentage of the main reinforcement steel required for positive moment and is determined from the following formulas:

For reinforcement perpendicular to traffic:

$$\text{Percentage} = 220/S^{1/2} \text{ (maximum 67\%)} \tag{3.20}$$

For reinforcement parallel to traffic:

$$\text{Percentage} = 100/S^{1/2} \text{ (maximum 50 \%)} \tag{3.21}$$

Where S is the effective span length

For example, if the area of main positive reinforcement is A_s, then the area of reinforcement distribution reinforcement perpendicular to traffic would be $(220/S^{1/2})A_s/100$ sq. in.

3.1.9 <u>Temperature and Shrinkage Reinforcement</u>

Temperature and shrinkage reinforcement should be provided to prevent shrinkage cracks and cracks due to stresses arising from temperature variations. The amount of this type of reinforcement per foot width of slab is obtained from the formula:

$A_{TS} = 0.0018tb$

Where:

t = slab thickness (in)

b = width of strip (in)

3.1.10 <u>Serviceability Requirements</u>

The serviceability requirement is satisfied by checking the fatigue stress limits, steel stress limits, and deflection, ensuring that these do not exceed certain values.

Fatigue Stress Limits:

The AASHTO code specifies that fatigue stress limits are not considered for decks designed by the AASHTO approximate method, where primary reinforcement is aligned perpendicular to traffic.

Steel Stress Limits:

Crack width control is implicitly provided by checking the following equation for stress at service load level:

$$f_s = \frac{z}{(d_c A)^{1/3}} \leq 0.6 f_y \qquad (3.22)$$

Where:

z is an exposure factor

d_c = thickness of concrete cover measured from extreme fiber to center of the closest bar or wire in inches

A = effective area in square inches of concrete surrounding the flexural tension reinforcement and having the same centroid as the reinforcement, divided by the number of bars or wires.

The quantity z should not exceed 170 kips per inch for members in moderate and a value of 130 kips per inch in severe exposure.

The stress in the reinforcement at service load for a cracked section is calculated using elastic theory. The stress in the reinforcement is determined from the following formula:

$$f_s = \frac{M}{A_s jd}$$

Where:

M = unfactored service load moment

d = distance of the extreme fiber from the center of reinforcement

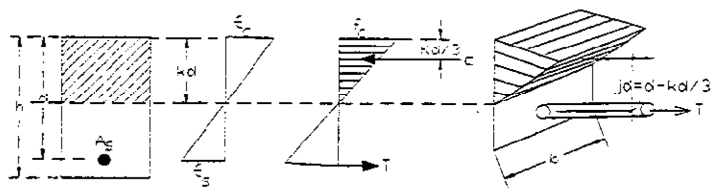
j = 1-k/3 (illustrated in Figure 3.2)

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

ρ = steel ratio

n = moduli ratio

The stress derived (f_s) should be less than the maximum allowable stress; otherwise, the section has to be re-designed. Figure 3-2 (a) to (d) shows the relationship between j, k, and d in the cracked section of the concrete.



(a) (b) (c) (d)

Figure 3.2 Internal forces in a single reinforced rectangular section (Hassoun 1998).

Control of Deflection:

Analysis for deflection can be avoided by choosing a minimum deck thickness base on the span of the slab and the type of structure supporting it. The standard practice is to use the relationships described in 3.1.1 to choose a minimum slab depth.

3.2 The Empirical Design Method (AASHTO LRFD 9.7)

The empirical design of concrete slab specified in section 9.7 of the AASHTO LRFD code is used in the design of isotropic bridge decks. Decks designed by this method are based on the premise that the deck slab will fail in punching shear rather than flexural failure. This method does not really require any analysis; a prescribed amount of reinforcement is placed in the concrete deck slab if certain conditions are satisfied. The conditions are similar to those set by the Ontario Highway Bridge Design Code. Details of the various aspects to be considered in the design of AASHTO decks are given in the following sections. This procedure does not apply to overhang; decks overhang are designed in accordance with the AASHTO code.

The design procedure does not involve a systematic procedure as the AASHTO design method; however, the procedure will be outlined similar to the AASHTO procedure in order to make comparing the methods easier, but Table 3.4 summarizes the differences between the two methods.

3.2.1 Slab Thickness

AASHTO code specifies a minimum slab depth of 7 inches plus a sacrificial wearing surface where applicable; however, PENNDOT specifies that the slab should be at least 8 inches thick, and an additional half-inch layer of concrete should be added to compensate for surface wearing. Two-and-a-half inches of concrete cover should be provided on top of the transverse bars, and a 1-inch cover should be provided on the bottom transverse bars.

The AASHTO code also specifies a minimum core depth of 4.0 inches, and the ratio of the effective length to design depth should be greater than 6.0 but should not exceed 18.0. The code also specifies that an overhang beyond the centerline of the outside girder should be at least 5.0 times the depth of the slab; this condition is satisfied if the overhang is at least 3.0 times the depth of the slab and a structurally continuous concrete barrier is made composite with the overhang.

3.2.2 Span Length

On slabs monolithic with beams or walls, the effective length of the slab is the face-to-face distance between the walls. However, for slabs supported on concrete or steel girders, the effective length of slab is taken as the distance between the flange overhang, which is the distance from the extreme flange tip to the face of the web, disregarding any fillet. When the spacing between the beams is not uniform, the determination of the effective length of slab has to be modified. The determination of the effective slab length is taken as the larger of the two distances, shown in Figure 3.3. (ASSHTO 9.7.2)

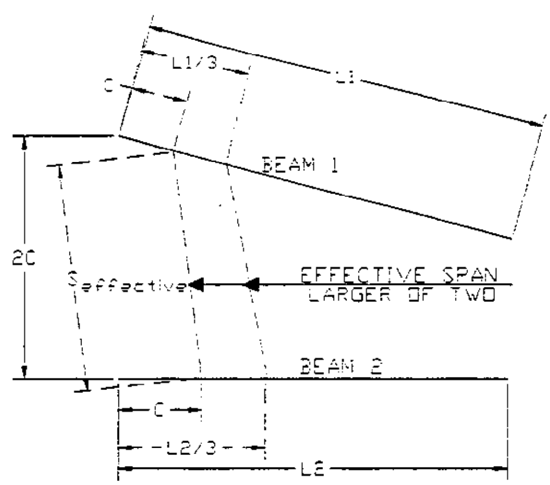


Figure 3.3 Effective length for non-uniform spacing of beams

3.2.3 Design Loads:

The isotropic decks are designed to support the loads of HS-25-44 (125% of HS–20-44), with the distribution in the load in accordance to the AASHTO code. A maximum of 30% of the live load is also factored into account for dynamic effects. Dead load tends to increase the shear capacity; therefore, it is not usually added as part of the design load.

3.2.4 Material Properties

The decks must also be fully cast in place and water cured, and it should be of uniform depth, except for haunches at girder flanges and other local thickening. In addition, the 28-day strength of the deck concrete should not be less than 4,000 psi.

3.2.5 Reinforcement

Where stay-in-place formwork is used, PENNDOT isotropic decks generally consist of four layers of reinforcement in which #4 bars are spaced 12 inches apart in both top and bottom transverse and longitudinal regions of the decks, as shown in Figure 3.4. However, depending on the skewness of the slabs, the deck reinforcement may be modified.

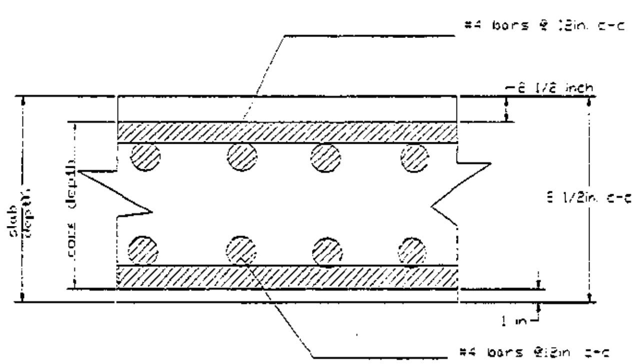


Figure 3.4 PENNDOT isotropic reinforcement pattern

Where steel girders are utilized, shear studs must be put in place to allow composite action of the concrete slab and the girders. Section 9.7.2 of the AASHTO LRFD code specifies a minimum of two shear studs at 24.0 inches on centers in the negative moment region of continuous steel structures. Where concrete girders are used, extension of stirrups into the deck is considered sufficient for shear requirements for the girder and slab to act compositely.

3.2.6 Details of Reinforcement:

The AASHTO LRFD design code specifies four layers of isotropic reinforcement for slabs designed using the empirical design method. The reinforcement shall be located as close to the outside surface as permitted by cover requirements. The reinforcement shall be provided in each face of the slab, with the outermost layer in the direction of the effective length. The minimum of reinforcement is $0.27 in^2/ft$ of steel for each bottom layer and $0.18 in^2/ft$ of steel for top layer, but the spacing between the bars shall not exceed 18.0 in. Prototype tests indicated that 0.2 percent reinforcement in each of four layers satisfies the strength requirement (AASHTO C9.7.2.5); however, a more conservative value of 0.3 percent of the gross area is specified for better crack control in the positive moment region. A lower amount of transverse reinforcement is specified for the negative moment regions over the girders; it has been observed that low stress exists in the negative moment regions in bridge decks (Allen 1991, AASHTO 9.7.2.5). The low amount of steel in the top layer also helps to reduce the tendency of spalling.

The four layers of reinforcement is placed in two mats. Generally, PENNDOT uses mats that comprise two layers of reinforcement in which #4 bars are placed in both the longitudinal and transverse directions. The longitudinal bars are placed parallel to the girders with 12-inch center-to-center spacing and transverse bars parallel to the center line of bearing with 12-inch x $(\sin\theta)^2$ center-to-center spacing for skew angles 70° or more. For skew angles (Figure 3.5) less than 70°, the longitudinal bars should be placed parallel to girders with 12-inch center-to-center spacing and the transverse bars should be placed normal to the centerline of the girders with 12-inch center-to-center spacing. For end deck slab regions, #4 bars should be provided at 6 inches center-to-center in the transverse direction, and #4 bars place 6 inches center-to-center in the longitudinal direction for skew angle less than 70°; all reinforcing bars should be epoxy coated. Further details of PENNDOT isotropic reinforcement schedule are shown in Figure 3.6 and tables 3.1 to 3.3.

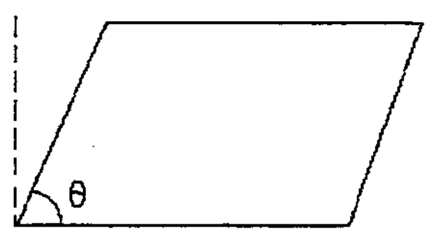


Figure 3.5 Diagram showing skew angle (θ) as defined by PENNDOT

S_o	** t (in)	Reinforcement Bar Spacing			
		Main Bridge Deck		Overhang Bridge Deck	
		Transverse Bars Each Layer S1 or S_2	Longitudinal Bars Each Layer S3 or S_4	Additional Transverse Bars top Layer $S_{2'}$	Additional longitudinal bars top layer $S_{4'}$
≤ 4'-0"	8.5	#4 @ d"	#4 @ 12.0"	# 5 @ d/2"	#6 @ 12.0"
≥ 4'-0" but ≤4'-9"	9.0	#4 @ d"	#4 @ 12.0"	# 6 @ d/2"	#7 @ 12.0"

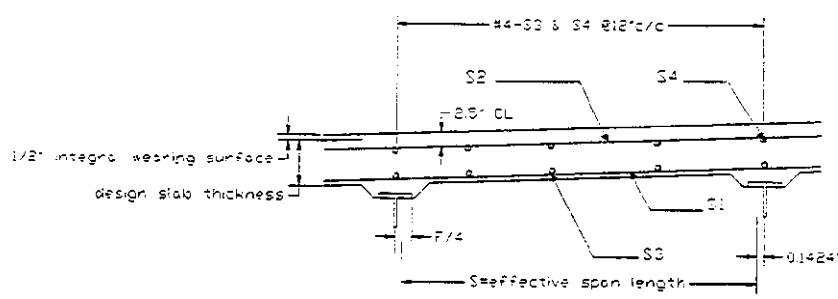


Figure 3.6 Typical cross section of an isotropic deck

Table 3.1 Deck Reinforcement schedule for skew > 70°

Table 3.2 Deck Reinforcement Schedule for skew angle 70° to 90°

S_o	** t (in)	Reinforcement Bar Spacing			
		Main Bridge Deck		Overhang Bridge Deck	
		Transverse Bars Each Layer S1 or S_2	Longitudinal Bars Each Layer S3 or S_4	Additional Transverse Bars top Layer $S_{2'}$	Additional longitudinal bars top layer $S_{4'}$
≤ 4'-0"	8.5	#4 @ 12.0"	#4 @ 12.0"	# 5 @ 6.0"	#6 @ 12.0"
≥ 4'-0" but ≤4'-9"	9.0	#4 @ 12.0"	#4 @ 12.0"	# 6 @ 6.0"	#7 @ 12.0"

** The values provided for t are minimum and are calculated as per AASHTO for corresponding S.

Table 3.3 Deck Reinforcement Schedule for skew angle less < 70°

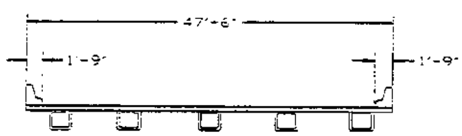
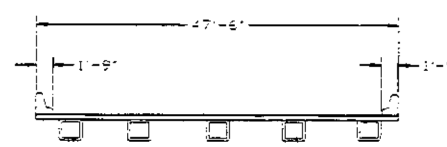
Table 4.2	
Skew angle ϕ	D
70° to 74°	11.0
80° to 89°	11.5
90°	12.0

3.2.7 Girder Spacing

In the design of isotropic decks, at least four laterally stiff girders must be present, and the spacing between them should not exceed 10 feet (Beal 1983). In decks supported on structurally stiff members, such as box beams, either intermediate diaphragms between the boxes are provided at a spacing not to exceed 25 feet, or the need for supplemental transverse bending between the box units is investigated, and reinforcement is provided if necessary. The transverse bending between box girders is illustrated in Figure 3.7.

Figure 3.7 Schematic of effect of relative displacement in torsionally stiff cross-section (AASHTO C9.7.2)

Table 3.4 Comparison of AASHTO and empirical design procedure

AASHTO Design Method:	**Isotropic Design Method:**
Given Structure 	**Given Structure** 
ASSHTO Design Method (continuation) Assumed Thickness t_{min}=[S+10]/30 Effective span length S=10-F/2 (for T beams or w sections) (AASHTO 8.9.2)	**Isotropic Design Method (continuation)** Assumed Thickness t_{min}=8.5" (PENNDOT)
Determination of Factored Loads Group I loading Group I = $\gamma[\beta.D+\beta_L(L+I)]$ Dead Loads: Factored weight of slab: $W_w=1.30w_ct/12$ Factored weight for future wearing surface $W_s=1.30w_ct_w/12$ Total dead load $W_D=w_W+w_S$	**Determination of Factored Loads** Decks designed in accordance with the empirical design are assumed sufficient to support HS 25 loading; no calculations are required.

<table>
<tr>
<td>
Factored weight of curb and parapet
$W_{p+c}=1.30A_{c+p}W_c$

Live Plus Impact Loads:

The 20 kip load from the HS 25 governs the design of the deck slab.
Maximum impact factor is 30%.
$I=50/(L+125)$
Where I is the impact fraction
L is the length of the span that is loaded to produce maximum stress in the member (AASHTO 3.8.2.1)
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<tr>
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ASSHTO Design Method (continuation)

Analysis of Moment

Continuous Span:
The positive and negative dead load moments are assumed to be
$M_D=w_DS^2/10$

The factored positive and negative live load plus impact moments are
$M_{L+I}=C_f[(S+2)/32]P_{L+I}$

Where C_f is a continuity factor applied to slabs continuous over three or more supports. C_f is assigned a value of 0.8.

Service load moment:

$\boldsymbol{M_U = M_D + M_{L+I}}$

Cantilever Spans:

$M_D=w_DS^2/2 + w_{c+p}L$

Width of wheel load for elements perpendicular to traffic
$E=0.8X + 3.75$ (AASHTO (3-17))
Moment per foot of slab is given by
$M=(P/E)X$
E should not exceed 7ft
</td>
<td>
Isotropic Design Method (continuation)

Analysis of Moments

Isotropic bridge decks are not analyzed to determine their moment capacity. Provided the conditions described in the preceding section for the design of isotropic decks are in place and the load is not above the design load described in the previous section, by the empirical design method, the deck is able to support the load.
</td>
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Total factored moment is $M_u = M_D + M_{L+I}$	
Design for Moment Materials: f'_c f_y Maximum and Min. Steel: $\rho_b = 0.85\beta\, f'_c/f_y$ $\rho_{max} = 0.75\, \rho_b$	**Design for Moments** *A* prescribed amount of steel reinforcement is placed in the deck slab provided that the deck has the right conditions for isotropic design. That value is $0.20 in^2/ft$ length of slab provided by isotropic design in each layer of the principal direction. Where the skew is less than 70°, the amount of reinforcement is doubled.
ASSHTO Design Procedure (continue) **Area of Positive and Negative Steel:** Reinforcement in the compression area of the slab will not be considered in the design. $NM_n = \phi A_s f_y(d-a/2)$ where $a = A_s f_y/(0.85 f'_c b)$ $d = h - d_w - d_c - D_{bar}/2$ **Checking Moment Capacity:** $a = A_s f_y/0.85 f'_c b$ $M_u = 0.9 A_s f_y(d-a/2)$	**Isotropic Design Procedure (continue)** **Checking Moment Capacity:** No standard method is in place to determine the moment capacity of the isotropic slab. Bridge deck slabs designed by the empirical design method are considered sufficient to support service, fatigue and fracture, and strength limit states requirements (BridgeSight 97). In fact, in accordance with the prescribed mode of failure, it is the shear capacity of the slab that is of greatest importance. There is no standard method for determining the shear capacity of such decks. However, some analytical methods have given reasonable predictions of the failure strength.
Checking Minimum Steel: $E_c = 33w^{1.5}(f'_c)^{1/2}$ $E_s = 290\,000$ ksi $n = E_s/E_c$ $y_t = [bh(h/2)+(n-1)(A_s d_c+1/2D_{bar})]/(bh+(n-1)A_s)$ $I_{cg} = (1/12)bh^3+bh(d')^2+(n-1)A_s(d'')^2$ where d' and d" are the distance of the center of the concrete and reinforcement from the center of gravity of the reinforced section, respectively.	**Checking Minimum Steel:** Isotropic decks are not analyzed for minimum steel. Steel reinforcement put in place in accordance with the empirical method is assumed sufficient. The minimum amount of reinforcement is given as $0.2 in^2/ft$ of slab.

$f_r = 7.5(f'_c)^{1/2}$ $M_{cr}=f_rI/y_t$ $1.2M_{cr} \le M_u$? (ok if true) **Distribution of Reinforcement:** (ASSHTO LRFD 9.7.3.2) Reinforcement is placed in the secondary direction in the bottom of the slabs as a percentage of the primary reinforcement for the positive moment as follows: **ASSHTO Design Procedure (continue)** Primary reinforcement parallel to traffic: $100/S^{1/2} \le 50$ percent For primary reinforcement perpendicular to traffic: $220/S^{1/2} \le 67$ percent. Where S is the effective span length	**Distribution of Reinforcement:** No distribution reinforcement is placed in isotropic decks. **Isotropic Design Procedure (continue)**
Serviceability Requirements Fatigue Stress Limits: Fatigue stress limits need not be considered for concrete deck slab with primary reinforcement perpendicular to traffic and designed in accordance with AASHTO approximate method. Stress in reinforcement at service load for cracked section: Crack control is implicitly controlled by the following requirement $f_s = z/(d_cA)^{1/3} \le 0.6f_y$ $f_s = M/A_sjd$ Service load moment (unfactored):	**Serviceability Requirements** Isotropic decks are assumed to meet the serviceability requirement for fatigue stress limit, steel stress limits, and deflection control.

$M=M_D+M_L$ $j = 1- k/3$ $k = (2\rho n + (\rho n)^2)^{1/2} - \rho n$ Control of Deflections: This can be satisfied by selecting a depth of slab that is greater than the minimum thickness required.	
ASSHTO Design Procedure (continue) **Notes:** Use same reinforcement in the cantilever spans Place transverse reinforcement in deck slab parallel to centerline of bearing for skew angles of 75° and more. For angles less than 75°, the bars shall be placed normal to centerline of the bridge.	**Isotropic Design Procedure (continue)** **Notes:** The empirical design method does not apply to cantilever slabs Place transverse reinforcement in deck slab parallel to centerline of bearing for skew angles of 70° and more. For angles less than 70°, the bars shall be placed normal to centerline of the bridge.

The above table shows the ease of the isotropic design method; "no analysis" is required. Because of the equal spacing of the reinforcement, contractors also prefer to use the isotropic design method. Unlike the conventional method, no moment capacity calculations are made, and the deck is not analyzed for serviceability or limit states; it is assumed that the isotropic deck meets the requirements for serviceability once other various criteria are met.

CHAPTER 4. APPLICATION OF THEORY

This chapter presents two examples of a deck designed by the standard AASHTO procedures and the same deck designed using the empirical design method. Both designs employ the method and procedures discussed in Chapter 3. The results are then compared.

4.1 AASHTO Deck Design

4.1.1 Problem:

Design the deck shown in Figure 4.1 given the following conditions:

- The cross section of the deck shown below (Figure 4.1)
- Bearing piers is 70° skewed
- The concrete slab has a 28-day strength of 4,000 psi.
- The slab is cast in place and water cured
- Assume a half-inch wearing surface
- The slab acts composite with the girders
- Parapet and curb are composite and structurally continuous with the overhang
- Deck slab is supported on prestressed concrete box girders
- Bridge is a single-simple span type
- Load of stay-in-place (SIP) forms (w_{SIP}) = 15 lb/ft
- Factored weight of parapet + curb = 1.3x3.37x15lb/ft^2
- Material properties: f'_c = 4000 psi f_y = 6,0000psi
- Simple span length 45 ft.

- Girder size 36 in. x 36 in.

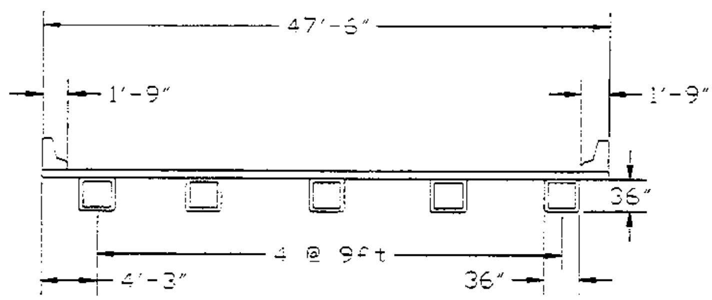


Figure 4.1 Typical cross section of bridge normal to alignment

4.1.2 Slab Thickness

Minimum slab thickness equals 7 inches, but use a slab thickness of 8.5 inches.

4.1.3 Dead Load

Girder spacing: S = 9.0 ft center to center

Skew angle (ϕ): 70°

Assumed slab thickness: t = 8.5 in.

Weight of concrete: $w_c = 150\ \text{lb/ft}^3$

$W_D = 1.3(w_c)(h)/b + 1.3w_{SIP}$

$W_D = 1.3\ [(9\ \text{in.})/12]150\ \text{lb/ft}^3 + 1.3\text{x}15\ \text{lb/ft}^2$

$= 146.25\ \text{lb/ft}^2 + 19.50\ \text{lb/ft}^2$

Total dead weight (W_D): $165.75\ \text{lb/ft}^2$

4.1.4 Live Load + Impact Load

$P_{L+I} = \gamma\beta_L(P+\text{Impact})$

$P_{L+I} = 1.3 \times (1.67) \times 20.8$ kips

$= 45.2$ kips

4.1.5 Live and Dead Load Moment

Perpendicular to traffic:

$$M_D = \frac{W_{DX}S^2}{10} \tag{4.1}$$

$= 0.146\text{x}(6)^2/10$

$= 0.5256$ k-ft/ft

$$M_{L+I} = \frac{C_f(S+2)}{32}P_{L+I} \tag{4.2}$$

$M_{L+I} = [0.8(6 + 2)/32] \times 45.2$

$= 9.04$ k-ft/ft

$$M_U = M_D + M_{L+I} \tag{4.3}$$

$M_U = 0.5256 + 9.04$

$= 9.56$ k-ft/ft

Moment Perpendicular to traffic: (AASHTO 3.24.3)

Live load moment (LLM) = 900S

= 6075 ft-lbs (for HS –20 loading)

For HS-25 LLM = (5/4) x HS-20

LLM = (5/4) x 6075 ft-lbs

$LLM \ = 7594$ ft-lbs/ft

$\rho_b = 0.31$ $\rho_{max} = 0.0232$

4.1.6 Positive Moment Reinforcement:

This example does not include reinforcement in compression face. #5 bars which have diameter of 0.625, d is as follows:

$$d = t\text{- bottom cover} - 1/2 \text{ Diameter} \tag{4.4}$$

$= 8.5$ in. $- 1$ in. $- (1/2)0.5$ in.

$= 7.25$ in.

$$a = \frac{A_S f_Y}{0.85 f_c^1} \tag{4.5}$$

$= 1.307\ A_s$

$$\frac{a}{2} = 0.654\ A_s$$

$$M_u \,x12 = 0.9 A_s f_y (d\text{-}a/2) \tag{4.6}$$

$10.55(12) = 0.9\ A_s(60)(d\text{-}0.654\ A_s)$

Therefore: A_s=0.30 in^2

Used # 5 bars spaced at 11 inches on center in slab $A_s = 0.30$ in^2

Area of positive reinforcement = 0.30 in^2/ft of slab (perpendicular to traffic)

$$\rho = \frac{A_S}{bd} \tag{4.7}$$

$\rho = 0.30/(12x7.25)$

$= 0.0034$

Main Reinforcement Parallel to Traffic

From the moment of 7594 ft-lbs/ft, the area of steel reinforcement required = 0.22 in^2/ft

4.1.7 Distribution Reinforcement

For distribution perpendicular to traffic:

Percentage = $220/S^{1/2}$

Percentage = $220/6.75^{1/2}$

Percentage = 85

Hence, use 67% of the positive moment reinforcement

Area of distribution reinforcement = 0.67x 0.30in^2/ft

= 0.201 in^2/ft

For distribution parallel to traffic

Percentage = $100/S^{1/2}$

Percentage = $100/6.75^{1/2}$

Percentage = 38.49

Hence, use 38.49 % of the positive moment reinforcement

Area of distribution reinforcement = 0.3849x 0.30in^2/ft

= 0.115 in^2/ft

4.1.8 Moment Capacity:

$$a = \frac{A_S f_Y}{0.85 f_c^1} \qquad (4.8)$$

a = 0.30(60)/[0.85x4.5x12]

a/2 = 0.196

$M_u = [0.9x0.30x60x(7.25\text{-}0.196)]/12$

$= 9.52$ k-ft/ft

Since the ultimate moment is 9.52 kip-ft/ft, the moment capacity is adequate.

4.1.9 Minimum Steel Check:

$E = 33w_c^{1.5}(f'_c)^{1/2}$

$E_c = 3834$ ksi $E_s = 29000$ksi

Where: $n= \dfrac{E_c}{E_S}$ (4.9)

$n=7.56$

$(n\text{-}1)A_s = (7.56\text{-}1)(0.30)$

$= 1.96 \text{ in}^2$

$[bh+(n\text{-}1)A_s]y_t = bd(h/2\ )+ (n\text{-}1)A_s(d_{ct}+D/2)$ (4.10)

Where: D_{bar} = bar diameter

d_c = concrete cover

y_t = distance from neutral axis to extreme top fiber

$$y_t = \frac{[bh(\frac{h}{2}) + (n-1)(A_S d_c + \frac{1}{2}D_{bar}))]}{bh + (n-1)A_S} \quad (4.11)$$

$y_t = [108(4.50) + 5.579]/(108+2.268)$

$y_t = 4.22$ in

$$I_{cg}= \frac{1}{12}bh^3+bh\,(d')^2+(n\text{-}1)A_s(d'')^2 \quad (4.12)$$

$I_{cg} = (1/12)(12x9^3) + 12x9(-0.223)+(7.56\text{-}1) \times 3.473^2$

$= 643 \text{ in}^4$

$f_r = 7.5\sqrt{f'c}$ (4.13)

$f_r = 7.5(4500)^{1/2}$

$= 503 \text{ psi}$

$$M_{cr} = \frac{f_r I_{cg}}{c} \qquad (4.14)$$

f_r = ultimate tensile stress of concrete

c = distance between the neutral axis and the fiber having maximum tensile strain

$M_{cr} = 0.503(643)/4.22(12)$

$= 6.386$ k-ft/ft

$1.2\ M_{cr} = 7.66$ k-ft/ft

Since the moment capacity (9.52 kip-ft/ft) is greater than 1.2 times the cracking moment, the moment capacity is sufficient.

4.1.10 Temperature and Shrinkage Reinforcement:

$A_{TS} = 0.0018tb$ (4.15)

$= 0.0018 \text{x} 8.5$ in. x 12 in.

$= 0.1836 \text{ in}^2/\text{ft}$

Where: A_{TS} = area of temperature and shrinkage steel

t = thickness of deck

b = width of slab

4.1.11 Serviceability:

Fatigue Stress Limit:

Fatigue stress limits need not be considered for concrete deck slab with primary reinforcement perpendicular to traffic and designed in accordance with AASHTO approximate method.

Distribution of flexural reinforcement

$A = \text{spacing}\ [((t\text{-cover}_t)/2) + \text{cover}_t]$

$z = 130$

$$f_s = \frac{z}{(d_c\,A)^{1/3}} \leq 0.6f_y \qquad (4.16)$$

$$f_s = 130/(2x49.5)^{1/3}$$

$$= 28.1 \text{ ksi}$$

z is given a value of 130 for members in severe condition, 170 for moderate conditions, and 100 for buried structures.

$$f_{smax} = 0.6f_y$$

$$= 36 \text{ ksi}$$

If $f_s < f_{smax}$, then the value is OK; otherwise, re-check reinforcement.

Moment at Service Load (unfactored):

$$M_L = \frac{C_f\,(S+2)}{32}P$$

$M_L = 5.845$ k-ft/ft

$$M_D = \frac{t}{12}\frac{w_c S^2}{10}$$

$M_D = 0.3825$ k-ft/ft

$M = M_D + M_L$

$M = 6.329$ k-ft/ft

Stress in Reinforcement at Service Load for Cracked Section:

$$\rho = \frac{A_S}{bd}$$

$k = (2pn + (pn)^2)^{1/2} - pn$

$k = 0.17$

$j = 0.943$

$j = 1 - k/3$

$f_s = M / A_s jd$

$$f_S = \frac{6.227x12}{0.338in\ x0.93x7.19} ksi$$

$f_s = 36.9$ ksi

If $f_s < f_{smax}$ design OK, otherwise re-design

Control of Deflection:

Minimum slab thickness for deflection control:

$t = 0.055S$

$= 0.055 \times 6.75 \times (12)$

$= 4.46$ in

Since the slab thickness chosen was much greater than that value, there is no need to analyze for deflection.

4.2 Empirical Design

4.2.1 Example Application of the Empirical Design Method

In this example, the deck in section 4.1 is designed using the empirical method of design. This design does not include the overhang; the overhang is designed in accordance with the conventional AASHTO design code.

Effective span length:

The effective span length is determined as per the AASHTO standard procedure. In this case, the effective span length is 6.0 feet.

Design Depth of slab:

Design depth = $h - t_w$

Where:

h = total depth

t_w = thickness of wearing surface

Design depth = 9 in. – 0.5 in.

= 8.5 in.

Core Depth:

The core depth can be easily determined from the following relationship:

Core depth = gross slab depth – top cover – bottom cover

Using a top cover of 2.5 inches and a bottom cover of 1 inch, the value of the core depth is:

Core depth = 8.5 in. - 2.5 in. - 1.0 in.

= 5 in.

Since the core depth is greater than 4.0 inches, it satisfies the code requirement for minimum core depth.

Check the design conditions:

The design must conform to the following conditions to satisfy the empirical design method. If not, then the deck should be designed using the conventional AASHTO design procedure; however, if these conditions are satisfied, then it is assumed that the deck

satisfies the service, fatigue, fracture, and strength limit states requirements for bridge deck subjected to the loads described in previous sections. It is important to note that these requirements do not apply to the deck overhang; the overhang should be designed in accordance with the conventional AASHTO design code. Table 4.1 shows a list of conditions for checking whether the decks meet the requirements for empirical procedures.

Reinforcement:

The slab shall be provided with two mats of reinforcement, with each face having a minimum reinforcement provided for shear and temperature stresses. Pennsylvania. DOT provides a minimum of 0.2 in^2/ft of reinforcement. This is accomplished by placing #4 bars at 12 inches center to center. The reinforcement should be epoxy coated, as indicated in

Chapter 3.

Table 4.1 Criteria for use of the empirical design method

Criteria	Satisfied (yes/No)
At least four torsionally stiff girders are in the cross section of the deck and are either concrete or steel (eg box girders)	Yes
The deck is at least 8.5 in. thick, including a 0.5 in wearing surface	Yes
The deck is fully cast in place	Yes
The spacing between girders does not exceed 10 feet	Yes
The deck slab and girders act compositely	Yes
The depth to effective span ratio does not exceed 18.0	Yes (Span-to-depth ratio = 6 ft x12/ 8.5 in. = 8.5)
There is an overhang beyond the centerline of the outside girder at least 5.0 times the depth of the slab, or the overhang is 3 times the depth of the slab and structurally continuous concrete barrier is made composite with the overhang	Yes (Overhang from centerline = 5 ft- 9 in.)
The specified 28-day strength of the concrete is at least 4,000 psi	Yes
The deck has uniform thickness except local thickening in areas such as over girders	Yes

Spacing of reinforcement:

Spacing = bar area/ area required

$= 0.20\ in^2/0.2\ in^2/ft$

$= 12$ in.

Using #4 bars, place bars 12 inches in the top and bottom longitudinal and transverse directions.

The reinforcement for the overhang is designed in accordance with the conventional AASHTO designed method, as illustrated in the previous example. Since the reinforcement is 70°, the reinforcement should be place in the direction of the skew in the skewed region of the deck. The spacing of reinforcement along the skewed area of the deck is determined from the following relationship:

$S_r = 12\sin^2(\theta)$ where θ is the skew angle. (4.17)

$S_r = 12*\sin^2(70°)$

$S_r = 10.6$ in.

Use $S_r = 10.5$ in.

Where:

S_r is the spacing of the reinforcement

Use #4 bars spaced at 10.5 inches center to center in the skewed region of the deck and three feet into the unskewed regions, as shown in Figure 4.3.

4.3 Comparison of design from Both Methods

Comparing the positive reinforcement, the isotropic design led to a reduction in the amount of main positive reinforcement by 41 percent. Although the isotropic deck is usually slightly thicker, it is much easier to make a thicker deck than to construct a deck with variation in steel reinforcement in top and bottom layers compared to one with isotropic reinforcement. It is also seen that the AASHTO design is much more complicated than the empirical design. No calculations are made to determine the moment capacity of the deck using the empirical design method, unlike the conventional

AASHTO design method. In addition, the isotropic deck designed using the empirical method is not analyzed for serviceability.

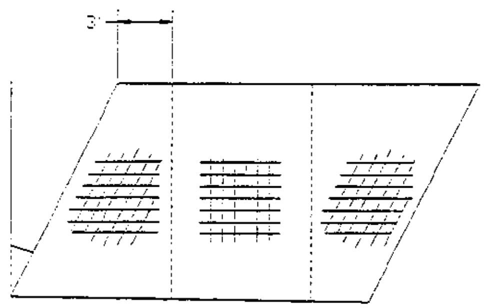


Figure 4.2 Reinforcement layout for skewed isotropic deck for skew angle > 70°

CHAPTER 5. RESULTS AND DISCUSSION

Results of the findings of the field investigations are presented in this chapter. Both qualitative and quantitative observations are shown. The results and the AASHTO and isotropic design procedure are also discussed.

5.1 Field Results for AASHTO and Isotropic Bridge Decks

During the first five years of investigation, little information was collected on the AASHTO designed decks, but photographs were taken for some of these decks, and condition reports were available. During the follow-up field investigation in June 2000, three AASHTO decks were visited together with their isotropic counterparts. Interestingly, the bridge with the most cracks was the AASHTO type located along SR 403 in Somerset County. Unlike most of the other bridges that were observed, it had four hair-line longitudinal cracks that extended from one end of the abutment to the other. Those cracks were located approximately over the edges of the two central box girders. Its isotropic counterpart in Blair County along SR 4027 exhibited less cracking, as shown in figures 5.1 to 5.5. Unlike its AASHTO counterpart, there was a small amount of transverse cracking, it was not extensive. At the time of the visit in June 2000, some of those cracks were sealed with asphalt, so, the severity of those cracks could not be observed. During the field visit in June 2000, the traffic volume on both decks was very low.

The isotropic deck in Venango County was in very good condition. Located

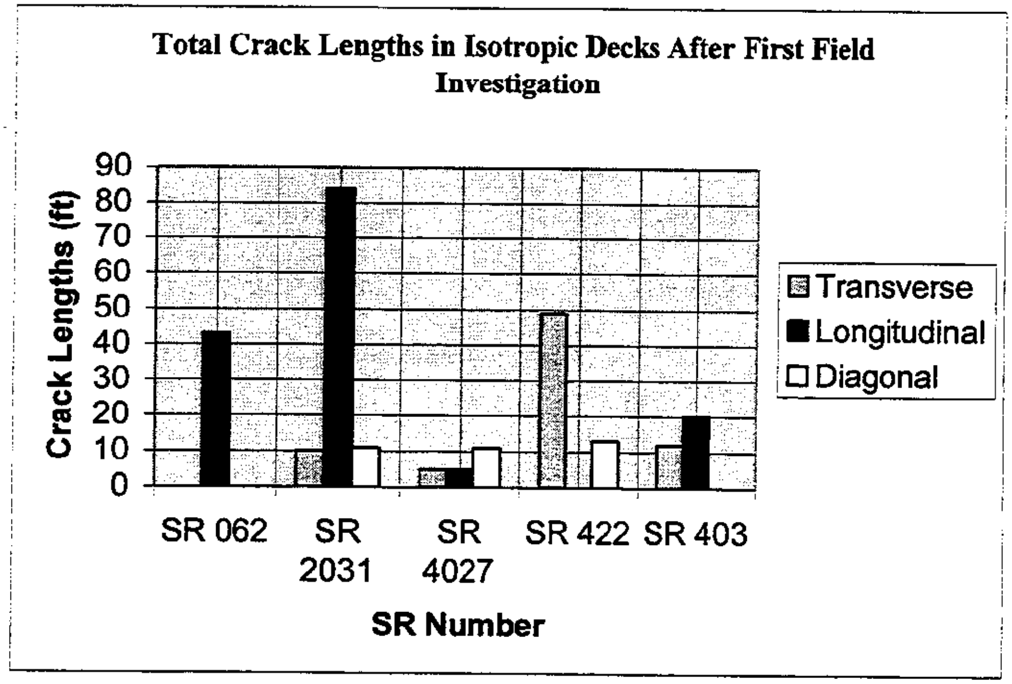


Figure 5.1 Crack distribution on some bridge decks in Pennsylvania

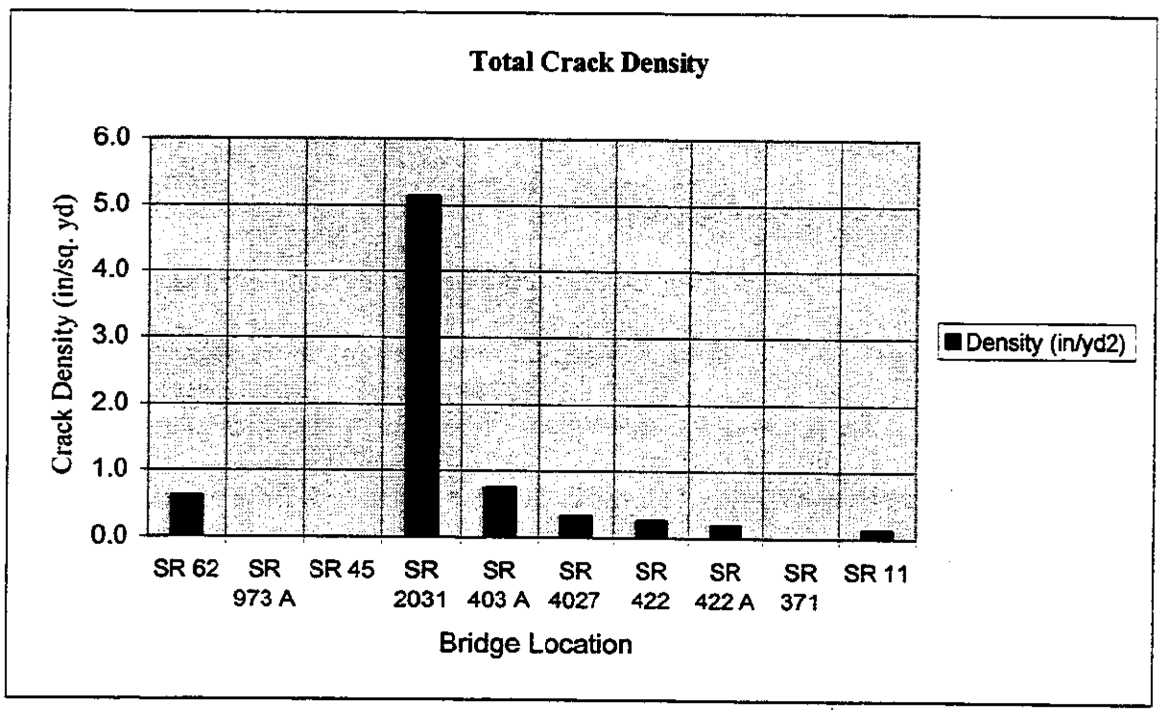


Figure 5.2 Total crack density in some of the decks during the field visit in 1994

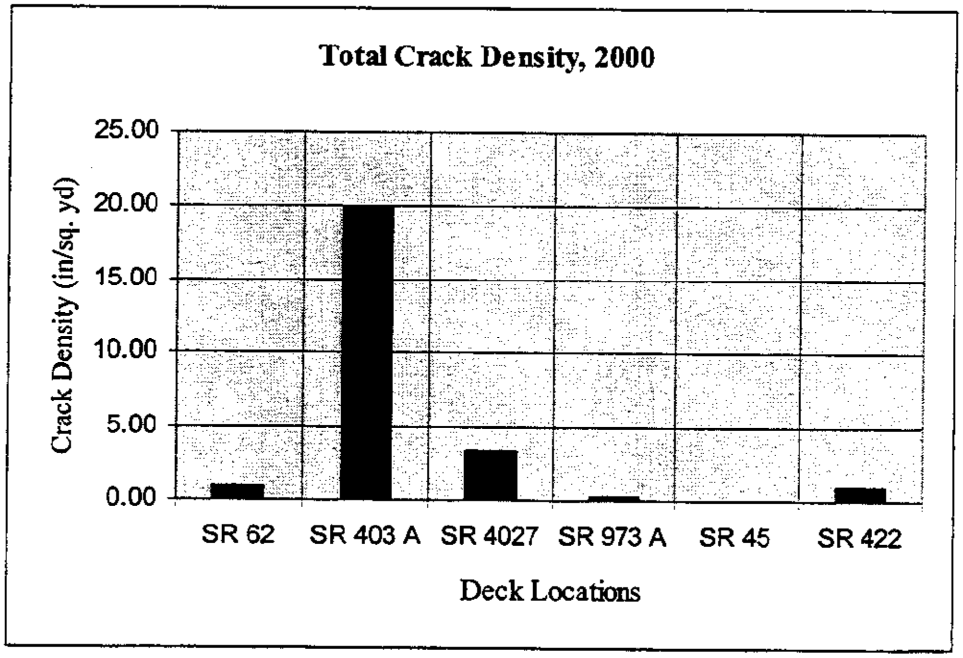


Figure 5.3 Total crack density in some of the decks during the field visit in 2000

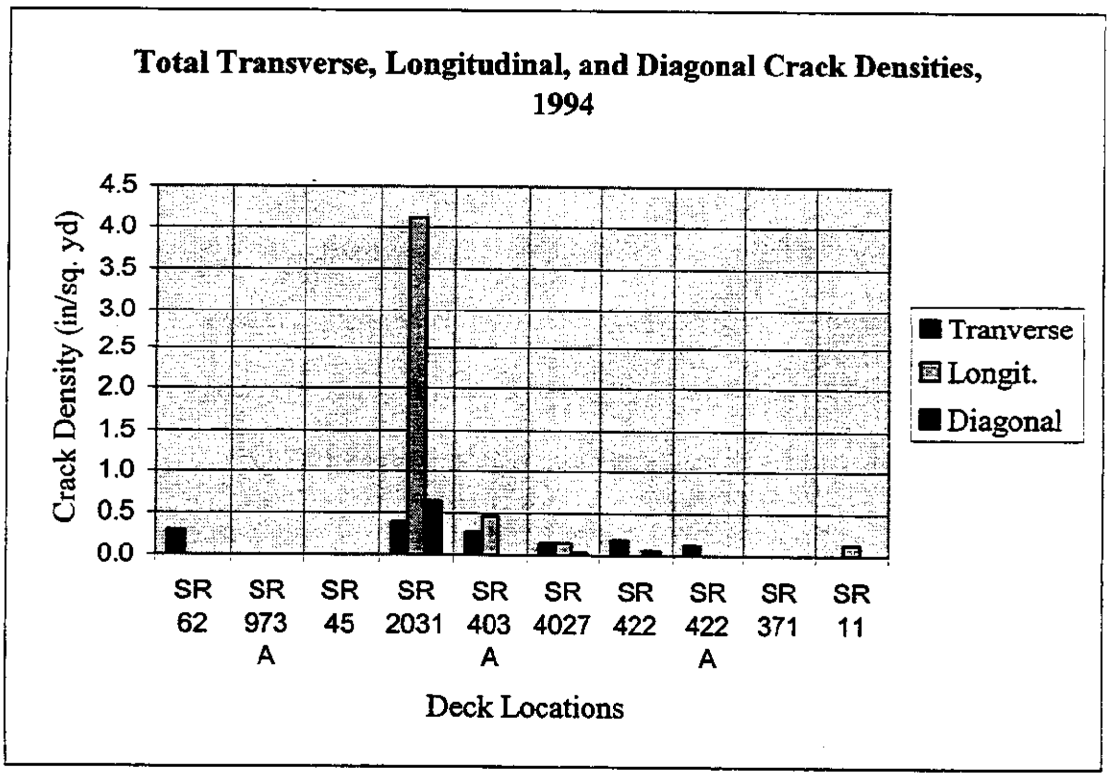


Figure 5.4 Total crack lengths in some of the deck in 1994

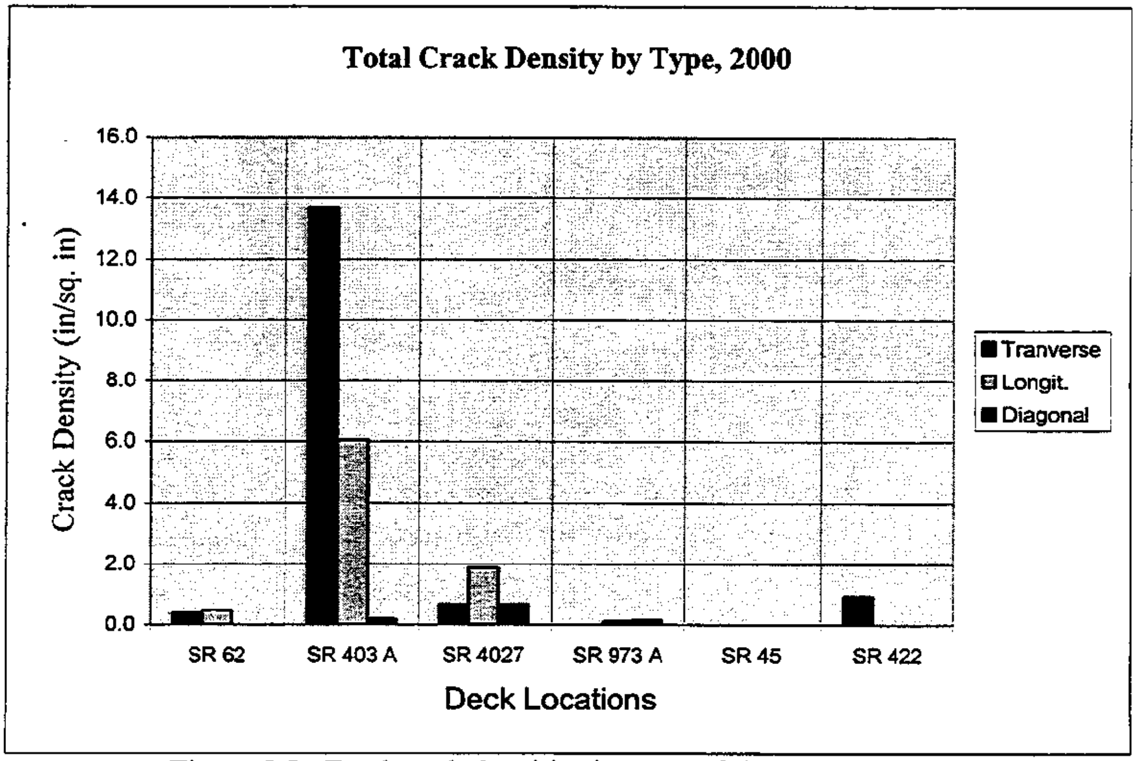


Figure 5.5 Total crack densities in some of the decks in 2000

immediately outside Hepburn Township, it supports traffic along one of the main entrances to the small town. The deck had some transverse cracks, but they were of minor severity. During the follow-up visit in June 2000, the deck was still in good condition, and although there were a few additional transverse cracks, they were still minor. In addition to the transverse cracks, two longitudinal cracks were observed in the central region of the deck that was not present during the visit in 1994.

District 3-0 decks were also in very good condition. The isotropic deck along SR 45 in Union County was in excellent condition; no cracks were observed during any field visit. During the follow-up visit in 2000, the deck was still in excellent condition. Four 4-inch cores were removed from the deck; however, results from testing these core samples were not available. During the visit in 2000, the traffic on the deck was moderate, with a significant portion of the vehicles being heavy trucks. The AASHTO counterpart located in Lycoming exhibited some cracks, but they were very few and of low severity. The bridge was also moderately traveled during the visit in 2000 but mainly by light vehicles. In addition to the few cracks, there was also a small amount of flaking of some of the aggregates from the AASHTO deck.

The isotropic and AASHTO decks along SR 422 in District 10-0 were located about a mile from each other. During the field visits, they were frequently traveled by a wide range of vehicles and may be the two bridges that experienced the most similar traffic volume. Field visits, including the follow-up visit, revealed only minor cracking. Interestingly, most of the cracks observed on both decks were located in approximately the same region of the decks, and the cracks were all transverse. Those cracks were found on the central spans of both decks, and the simple outer spans on both types of decks exhibited virtually no cracks.

Field reports stated that the decks in Berks County did not suffer any severe cracking. A photograph of the deck at Berks (Figure 2.25) revealed extensive longitudinal cracking, but they were reported to be minor. No photographs of its AASHTO counterpart were available, and those decks were not visited during the follow-up visit.

Field reports also stated that the decks in Susquehanna and Wyoming were in good condition during the field visits. Photographs in 1994 showed that the isotropic decks were in very good condition. No cracks were reported in the field report except for the deck in Susquehanna that had a longitudinal crack along the entire length of the deck (Figure 2-26). The crack seemed to have propagated from a construction joint at the center of the deck slab. Field reports showed that the other isotropic deck in District 4-0 along SR 371 was in excellent condition, with no signs of cracks (Figure 2-27); however, no reports were available for the AASHTO decks in that district.

Figures 5.6 to 5.7 shows the percentage distribution of the three types of cracks on the bridge decks.

5.2 Discussion

5.2.1 Discussion of Results

From a qualitative point of view, the isotropic decks have performed similar to the AASHTO-type decks. In some cases, both the isotropic and AASHTO decks revealed

very few, or virtually no, cracks, and none of the decks showed any sign of spalling. This was especially true for the short single-span decks; the majority of the short-span isotropic decks showed very few, or no visible, cracks. For instance, isotropic deck located in Union County and Wayne County, along SR 45 and SR 371, respectively, had no visible

cracks during the field visits. The short simple-span sections of the decks in District 10-0 along SR 422 also showed virtually no cracks during the field visits; the incidence of cracks appears to increase with increasing number of spans, as is common to all different types of bridges. During the 1994 visit, both bridge decks along SR 422 had cracks along the same region, the central spans.

Generally, the isotropic bridge decks did not exhibit any visible difference when compared to their AASHTO counterpart. In fact, they appeared to behave more similarly than dissimilarly. For example, the bridges in District 3-0 on SR 45 and SR 973 both showed very low crack density. Cracks were observed on the AASHTO deck in that dis-

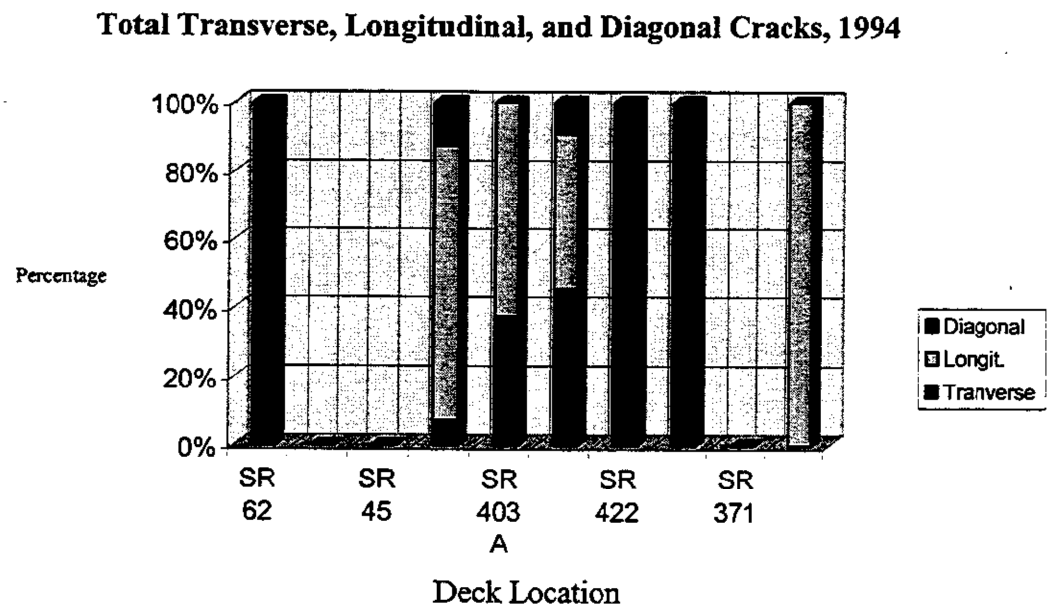


Figure 5.6 Percentage distribution of cracks on some of the deck in 1994

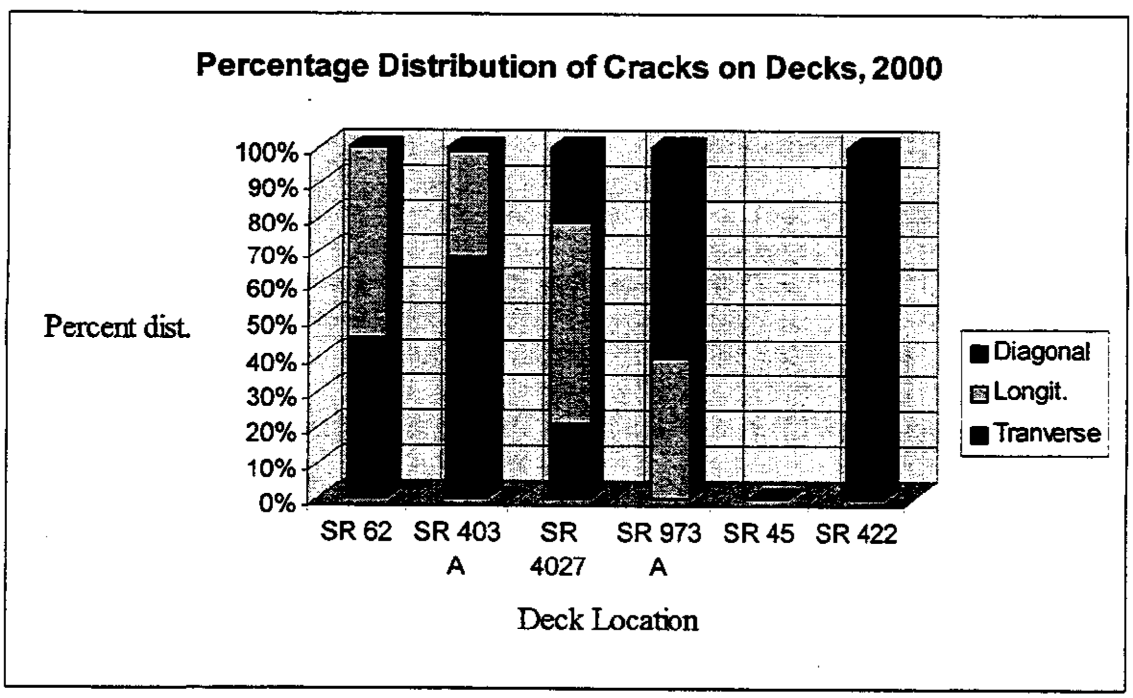


Figure 5.7 Percentage distribution of cracks some decks in 2000

trict, but they were very few, as revealed in figures 5.2 and 5.4; no cracks were observed on the isotropic deck. However, there was a slightly higher percentage of longitudinal cracking in the isotropic decks, as seen in figures 5.2 to 5.7, with the exception of the AASHTO deck along SR 403.

Data collected earlier, as seen in Figure 5.1, shows that there was a relatively high percentage of longitudinal cracking present in both the isotropic and AASHTO decks. Although the longitudinal cracks could have resulted from flexural cracking imposed by wheel loads, other factors could have influenced this type of cracking. Two possible factors are construction practices and weather conditions, as pointed out by Kraus et al (1996). There could have been flexural cracking over the girders because of the low amount of reinforcement as revealed by Allen (1991). The stiffness of the girders during the setting stage of the concrete slab could have also initiated the longitudinal cracks. This phenomenon may arise if the slab in the clear span region did not have a very stiff support during the setting stage. Consequently, longitudinal flexural cracking could have resulted approximately at the point of the slab just over the edge of the box girders. This phenomenon may also be explained by relative displacement of adjacent girders, as depicted in Figure 3.7; it appears that the cracks were being formed at the point of flexure. The relative displacement may be used to explain the unusual amount of longitudinal cracking that is taking place along the isotropic deck on SR 2031 in Berks County and the AASHTO deck on SR 403 in Somerset County. Another possible cause of the relatively high amount of longitudinal cracking taking place in the decks on SR 403 and 2031 is overload, however, there was no record of permit loading for these two decks.

There is a concern for the extent of longitudinal cracking that is taking place in isotropic decks compared to diagonal or transverse cracks. If flexure is dominating isotropic decks, then the decks might have a short service life because the reinforcement in the decks does not meet the requirement for flexure by the AASHTO code. The fact that flexural cracks may be taking place may also be attributed to the thickness of the deck slabs. Allen (1991) has pointed out that the isotropic bridge decks that he observed in Ontario, Canada, showed significantly less flexural cracking than those in the United States. One prominent difference between the decks in the two countries is that those in Canada were at least 9 inches thick, while those he observed in the United States had an average thickness of 8 inches It can be shown that a small increase in the deck slab thickness can significantly increase its cracking resistance because the cracking moment of the plane concrete is directly proportional to the square of the thickness of the slab (Equation 2.1). Hence, flexural cracking can be reduced by using thicker deck slabs. However, Fang et al (1990) showed the bridge decks possess considerable strength even with extensive cracks and that it is in that stage the compressive membrane forces are activated, enhancing the load-carrying capacity of the decks. During the follow-up field visit in 2000 there was an increase in the amount of transverse cracks, but the relative distribution of the different types of cracks did not change significantly except on SR 403 and SR 62, where the amount of transverse cracks and longitudinal cracks increased significantly relative to the longitudinal cracks and transverse cracks, respectively. The change in crack distribution in some of the decks between 1994 and 2000 is shown in figures 5.6 to 5.7.

Similar to longitudinal cracks, there was no distinct difference between the transverse cracks in isotropic and AASHTO-type decks; however, there was a slightly higher amount of transverse cracking in the conventional AASHTO decks. This difference could have resulted from many different factors other than the reinforcement pattern, as pointed out by Krauss (1996). Fang et al (1988), however, pointed out that transverse cracks were more prevalent in AASHTO-type decks due to the type of reinforcement; the higher amount of steel in the transverse direction of the deck increases the stiffness of the deck in that direction, subjecting transverse direction to greater stress. Hence, a higher amount of transverse cracks could have resulted.

Another important aspect of the bridges under investigation was that there was a considerable variation between the isotropic decks and their AASHTO counterpart; that is, the AASHTO control deck exhibited many variations apart from just the reinforcement pattern when compared to their isotropic counterpart. Although the decks were designed to perform similarly, variations in design, environment, and, possibly, material could have accounted for the difference in results. Krauss (1996) has shown how various factors affect the transverse cracking in bridge decks. A more reliable comparison could be made by placing isotropic and an AASHTO-type reinforcement pattern on a multi-span bridge together. In this case, the bridge could be comprised of a series of simple spans. Consequently, decks under such conditions would be almost identical in all aspect except the type of reinforcement. The Michigan Department of Transportation has successfully adopted this type of construction on three bridges constructed for the purpose of comparing the performance of isotropic and AASHTO decks.

Reports from NYDOT, MDOT, and ODOT all showed that their isotropic decks are performing satisfactorily. This gives more credence to support the findings of the performance of the PENNDOT isotropic decks. In addition, the total average crack density of the isotropic decks was 0.28 in./sq. yd., which was significantly less than the decks reported by NY DOT for similar time period. (The average transverse density for the NY DOT decks was 1.61 in/sq. yd). The only report in the literature review that did not support the usage of isotropic deck was that put forward by Allen (1991). All of the isotropic decks he observed were supported on steel girders, and Allen clearly stated that his findings might not necessarily apply to decks supported on stiff girders, such as box girders.

The empirical method used to design isotropic decks is a much simpler method than the standard AASHTO design procedure. Not only is the method simpler, but the reinforcement pattern is also simpler to implement during construction. Consequently, construction of these decks is expected to be faster and easier to assemble. This is likely to result in lower construction cost. The Michigan Department of Transportation has already reported that contractors favor the isotropic deck design over the standard AASHTO deck because of the ease of construction. The lower amount of reinforcement in isotropic decks will also reduce the tendency of spalling; decks with higher percentage of reinforcement are more prone to spalling. Higher amount of reinforcement implies that more steel may be exposed to corrosive agents; hence, higher amounts of rusting may be present in AASHTO decks that may result in the threshold pressure necessary for spalling to occur.

5.2.2 Cost Evaluation of Isotropic Decks verses AASHTO Decks

As seen in Table 2.13, in all cases, isotropic bridge decks had a reasonable savings over their AASHTO counterparts. Although some of these data were estimated rather than obtained from the actual cost of the decks, it is obvious that the savings may be due to the significant reduction in the amount of steel reinforcement that they utilize. However, the cost difference mentioned is only for initial testing, but the most important cost should take the life of the bridge decks into consideration, together with all the maintenance costs during its useful life. It is, therefore, premature at this stage to judge the two different types of decks based on overall performance from a cost perspective. If the isotropic decks continue to perform well, then the cost benefits could be significant.

Although a lower percentage of steel may enhance the life of the isotropic deck by making it more resistant to corrosive agents and reduce the tendency of spalling, the onset of flexural cracking may cause the bridge to deteriorate rapidly, hence, its life span. If that happens, the overall cost could be comparable to the AASHTO deck, or maybe more costly.

CHAPTER 6. CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

Based on this research the following conclusions are drawn:

1. The isotropic decks performed as well as the standard AASHTO-type decks designed according to current AASHTO standards.
2. There was no marked difference between the crack pattern on isotropic decks and AASHTO-type decks observed in the field. Crack pattern on both decks appears to be a characteristic of individual deck. Therefore, the isotropic reinforcement layout did not have a significant influence on the type of cracks present on the decks.
3. Simple-span isotropic and AASHTO-type decks showed considerably less cracking that decks having two or more spans.
4. The isotropic decks had a slightly higher longitudinal crack density than transverse or diagonal crack densities.
5. Diagonal cracks were more prevalent in the skewed decks. This is consistent with theory since these decks are subjected to more torsion.
6. The empirical design method used to design isotropic decks involves a much simpler procedure than the conventional AASHTO design procedure.

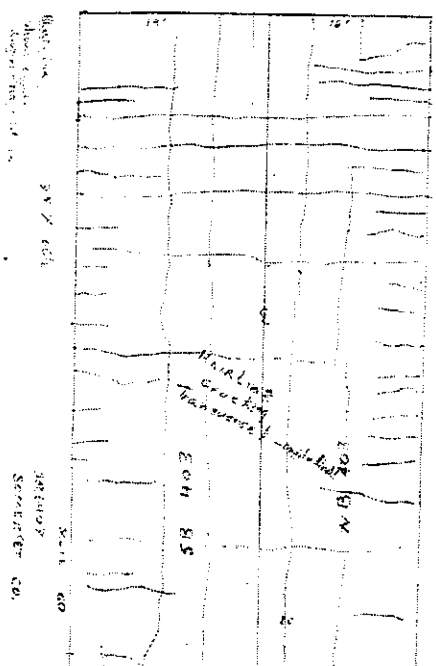
6.2 Recommendations:

- It is recommended that PENNDOT perform extensive fieldwork, collecting data in a more systematic manner in order to make distinctive effects of isotropic deck reinforcement on the performance of bridge decks. The following data collection is recommended: crack lengths, permit loads, cost of each deck, and the date of each investigation. In addition, the condition at the time of construction should also be documented. This should include weather condition, times of casting, curing period and method, and finishing procedures in accordance to factors affecting cracking pointed out by Krauss (1996). More detailed factors could be added, as seen in Table 2.1. A standard data sheet should also be formulated in order to maintain consistency in data collection.

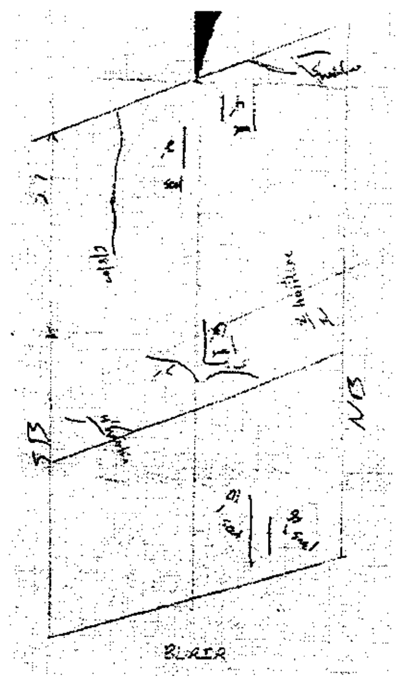
- The choice of bridge should also be more appropriately chosen. A number of bridges seem to deviate considerably from their counterparts in terms of their structure. In constructing two-way bridges, one direction could be constructed with AASHTO-type reinforcement and the other with isotropic. They would likely experience similar traffic volume and the same climatic condition. However, the closest counterparts could, perhaps, be obtained from the method adopted by the Michigan Department of Transportation in which isotropic decks and AASHTO decks were constructed on multi-span bridges. In this case, a series of simple-span decks were constructed on the same bridge and had similar span lengths.

- Although crack lengths were recorded, the data were not recorded in the same manner for each deck. The crack length is important in determining the crack density, which is more important than crack width for corrosion to control, as pointed out by Gerwick and Mehta (1982).
- The method adopted by NY DOT for the collection of crack lengths could also be adopted. NY DOT employed an automated means of collecting the data: from a scaled drawing, a computer program was used to automatically scan the sketches and record the total lengths of lines (cracks).
- Although permit loads were recorded, it was stated that some of these loads could have been missed due to inadequate monitoring. Therefore, a more reliable means of collecting these data should be adopted in future investigations.
- Cost data is missing from the collected information. Since cost was a major driving force behind the experiment of implementing isotropic bridge decks, it is important that the cost of each deck be recorded.
- Further investigation should be performed to find out the cause of longitudinal cracking during the early life of the deck and develop remedial measures to remedy or curtail the problem.
- Studies by Bakht and Mufti (1996) have shown that the bottom reinforcement is important for the development of arching action; therefore, in addition to temperature and shear reinforcement, additional reinforcement has to be considered in developing the necessary lateral force for the internal arch.

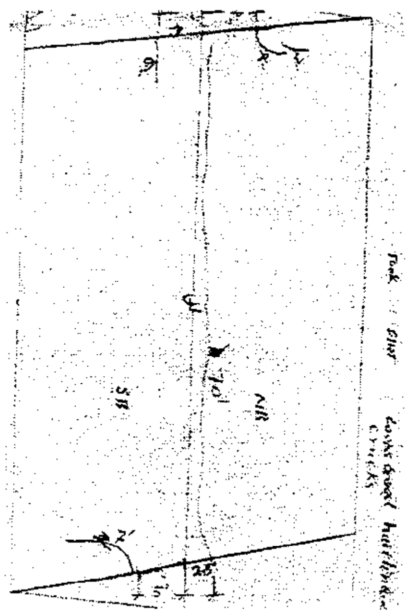
Appendix



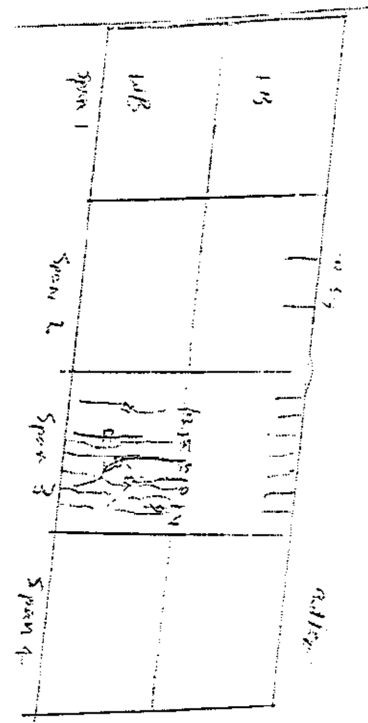
SR 403 AASHTO



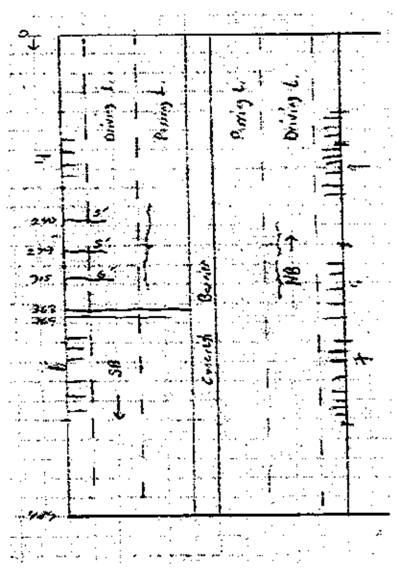
SR 4027 Isotropic



SR 2031 Isotropic



SR 422 Isotropic



SR 62 Isotropic

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