CHAPTER 2 LITERATURE REVIEW

2.1 Historical Review of Bridge Deck Analyses

A slab deck is structurally continuous in the two dimensions of the plane of the slab so that an applied load is supported by two-dimensional distributions of shear forces, moments and torques. The assessment of in-plane forces on the load carrying capacity of reinforced concrete slabs on steel I-girders has been recognized as a major field of bridge engineering for several decades. Research in this field originally concentrated on a study of the behavior of continuous beam supported by rigid girders concrete slabs. Great efforts have been made to develop simplified methods for live load distribution calculation. Some of them have been focused on bending moments in deck slabs. The early well-known on this research field is originally based on the work of Westergaard in 1930. In his analytical procedure, the deck was designed as a continuous beam supported by rigid girders, which are unable to deflect vertically. It is generally acknowledged that Westergaard's model yields conservative designs relative to strength and safety. It should be noted that this earliest renowned study was adopted in AASHTO Standard Specifications (1935) for computing the maximum transverse negative and positive moments in deck slab due to live load (M_{LL}^{-} and M_{LL}^{+}) per unit width. In the late 1940s, a study on I-beam bridges was performed by Newmark (1948). He recommended using slab design moments which are 30% lower than the theoretical design moment calculated in his research because of an effect of additional reserve of strength. His study also recognized that the strength enhancement due to compression membrane action occurred only after yield and that eventual collapse took the form of punch-out shear.

At the end of 1975, the Ontario Ministry of Transportation and Communications of Canada decided to develop an own national code for designing highway bridge decks, Ontario Highway Bridge Design Code (OHBDC). A series of tests were undertaken by academic researchers and the Ministry's Research and Development Division. Results showed that large reserves of strength existed in deck slabs under static and fatigue loading. This research work was supplemented by field tests of actual bridges. It was concluded that a slab's load carrying capacity was increased by in-plane restraints. Based on these findings, an empirical design method was proposed, involving an isotropic reinforcement layout in the deck. It should be noted that required reinforcement is considerably less than that specified by the AASHTO Code.

As a breakthrough in research knowledge, traditional codes of practice for bridge deck designs were developed with time and included in several national design codes such as AASHTO Specifications, Canadian Highway Bridge Design Code (CHBDC) and British Highways Agency Standard (BD81/02, 2002). In general, the AASHTO bridge deck design method assumes that slab-on-girder deck sections behave as one-way slabs acting in the transverse direction (perpendicular to traffic). In addition, nowadays AASHTO slab moment formula is based on a simply-supported slab subjected to a wheel load, and it is specified as *80%* as continuity factor of those bending moments when continuous deck slabs are placed on three or more supporting girders.

According to the design viewpoint, a well-recognized design tool, AASHTO Load and Resistance Factor Design of Bridge Design (LRFD, 2004), has provided two methods for deck slab design. The first, called the, Traditional Design Method" in Article 9.7.3, is typically referred to as the equivalent strip method. The second method is called the "Empirical Design Method" shown in Article 9.7.2 and relies on the arching action of concrete between the supports. To be simplified, AASHTO LRFD (2004) has provided Table A4-1 to determine the maximum M_{LL} based on several values of S by including impact factor IM of *1.33* and has offered multiple presence factors $\boldsymbol{m}$ of 1.20, *1.00* and *0.85* for 1-lane, 2-lane and 3-lane loading, respectively. To be able to attain the actual bridge deck behavior, some proposed reinforcement ratios for the bridge deck were recommended and verified using FEA to meet the strength and serviceability requirements of the design codes.

More recent research has sought to make deck design more efficient by more accurately modeling the actual load carrying mechanisms. The discovery of a mechanism called "internal arching" also offers designers and researchers greater the ability to develop strong designs with a lesser amount of reinforcing steel than is required in conventionally designed decks. In an attempt to mitigate damage as well as improve efficiency, reductions in the amount of top reinforcing steel has become a popular concept. Reducing or removing steel, especially in the upper layer, reduces the susceptibility of the deck to damage from steel corrosion and the subsequent debonding and spalling of the concrete. The primary design guideline requires a minimum steel area of the concrete area in both the top and bottom reinforcing mats to help control cracking and maintain confinement within the deck. Based on component testing and FEA, AASHTO (2000) has included provisions for the 'empirical design', which is similar to the Ontario deck design (OHBDC, 1991), except that a further reduction in the top layer of reinforcing steel is allowable.

Bridge deck deterioration is one of the most common deficiencies in a bridge system. Concrete bridge decks deteriorate faster than any other bridge component because of direct exposure to the environment, deicing chemicals, and ever-increasing traffic loads. Predicted increases in the number of trucks and axle loads on the roads will continue to degrade the roads more rapidly. Deterioration of the bridges is expected. Cao (1996) cited that about one-third of the nation's bridges in the United States have deficient decks. In this regard, one of the major problems of bridge decks face is corrosion of the reinforcing steel. This damage is generally initiated by cracking of the concrete in the top of the deck. The cracks allow moisture, deicing chemicals, and air to reach the steel and begin the oxidation process. When steel corrodes, it de-bonds from the concrete, thereby losing the structural value of the material's composite nature.

In general, magnitude of deck cracking and de-lamination due to corrosion are a major problem when measured in terms of rehabilitation costs and traffic disruption. To overcome corrosion-related problems several treatments can be conducted by reducing amounts of reinforcement (Cao, 1996), protecting corrosion steel reinforcement, or replacing reinforcement with alternative non-corroding materials. The CHBDC Code includes a new section (Section 16) for using fiber-reinforced polymer (FRP) composite bars as non-prestressed and prestressed reinforcements for concrete bridges (slabs, girders, and barrier walls).

Furthermore, due to the expansive nature of the corrosion products, and the relatively low tensile strength of concrete, portions of the deck can subsequently spall as the corrosive activity proceeds. Possible solutions to this problem are under investigation and include the use of epoxy-coated rebar, FRP reinforcement, fiber-reinforced concretes (FRC), low-permeability concretes and lower amounts of reinforcement. To date, full-scale field testing of these damage mitigation techniques has been sparse.

It should be noted that the above mentioned issue plays a significant role for the new era of bridge deck studies. Research in this field was continually conducted by several methods. Notably, after 1940s, bridge deck analysis had undergone major changes due to the digital computer advent roughly in 1950s to 1980s. Consequently, the refined methods of analysis such as grillage analogy, orthotropic plate, semi-continuum, and finite element methods were used in spite of the classical analytical method. An overview of previous works on related topics that provide the necessary background for the purpose of this research is provided hereinafter.

2.2 Analysis of Bridge Deck

Techniques used in the analysis and design of slab-on-girder bridges have improved more and more. Available theoretical methods are varied in their approaches as well as their accuracy and assumptions. Bridge superstructure can be idealized for theoretical analysis in many different ways. The different assumptions used in the formulation and calculations can lead to significant differences in the accuracy of the results. The major analytical approaches reported in the literature can be described as follows:

2.2.1 Approximate Methods

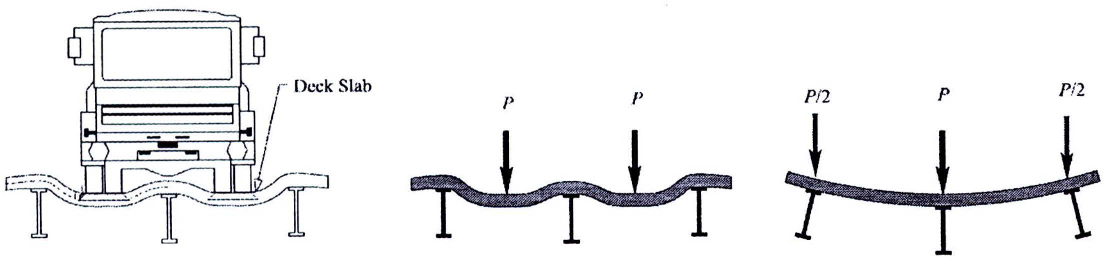
Traditionally, deck slabs have been analyzed using approximate methods. The approximate methods are based on calculating moments per unit width of the deck and design the reinforcement to resist these moments. This approach has been used successfully for many decades. However, the approximate methods were generally based on laboratory testing and/or refined analysis of typical decks supported on parallel girders and no skews. In case of deck slabs with unusual geometry, such as sharply skewed decks, the results of the approximate methods may not be accurate. For example, negative moments may develop at the acute corner of a sharply skewed deck. These moments are not accounted for in the approximate methods as they rely on assuming that the deck is behaving as a continuous beam.

Westergaard (1930) provides the basis for the design moments in bridge slabs due to concentrated loads, according to Section 3.24 in the AASHTO Standard (2002). Closed form solutions applied to homogeneous elastic slabs are developed by the mathematical theory of elasticity. These solutions are applied to bridge decks for common wheel loading from trucks.

Erps et al. (1937) developed simplified formulas for use in a design office, based on Westergaard's solutions. It is noted that Erps used side by side trucks with a center to center of wheels spaced at 0.914 m (3 ft) for his analysis. A wheel load, ***P***, equal to 53.4 kN (12 kip) was used for H-15 loading and 71.2 kN (16 kip) was used for H-20 loading. Also, Erps used 0.381 m (1.25 ft) for the diameter of circle over which the wheel load *P* is considered uniformly distributed when applying Westergaad's formulas.

Pucher (1977) established the sets of 13 Figures and 93 Charts of influence surface of moments for a single panel of isotropic slab with various shapes and support conditions. The Charts were useful to determine critical design moments in simply supported right slab deck. However, few simplified formulas or proposed guideline charts for transverse flexural designs in deck slabs due to affect of yielding of girder which would result in reducing safety factor has not general available. Therefore, the appropriate modeling technique of deck behavior should be investigated and established the guideline charts.

Bakht and Jaeger (1987) explored when a point load acts on a beam, the beam deflects and bends the slab transversely. The load is then transferred and shared out to the neighboring beams. Consequently, the continuity factor (***0.80***) might produce an over-estimation of M_{LL}^{-} and under-estimation of the positive bending moment at mid span of the slabs. In order to design a superstructure of bridge in transverse direction under wheel loads, the global behavior and local action need to be analyzed. They also proposed deck flexural responses due to one truck effecting the slab moments of a short span highway bridge (L < 30 m) as shown in Figure 2.1 (Bakht and Jaeger, 1987; Cao et al., 1995).



(a) Overall deformation (b) Local deformation (c) Global deformation

Figure 2.1 Deck flexural responses due to one truck effecting the slab moments: (a) Overall deformation; (b) Local (Primary) deformation; (c) Global (Secondary) deformation

However, research work carried out by Bakht and Jaeger (1987) as shown in Figure 2.1 is not taken into consideration in the Specifications. The local bending moment (M_l) can occur when slabs bend and act as main beams deflected under the loads as depicted in Figure 2.1(b). As it was found by this research, the global deformations (Figure 2.1(c)) directly affect on the overall bending slab moment (M_t) showed in Figure 2.1(a), which can greatly reduce the top reinforcements of the deck slab. The global bending moment (M_g) in Figure 2.1(c) is based on a longitudinal stiffness of the composite system (D_y).

Note that behavior of the deck slab system as a one-way slab is when its length exceeds its width (Hambly 1991). The basic difference between one way and two way slab is load distribution which is depends on support condition and span lengths. Mainly shorter span is carrying larger load than longer span for 1-lane loading (see Figure 2.1). As can be noticed from Live Load (LL) design considerations in AASHTO LRFD (2004), the maximum provided multiple presence factors $\boldsymbol{m}$ of 1.20 is for 1-lane loading $\boldsymbol{M_{LL}}$. Some concerning research works were very good but limited. Cao (1996) has ignored the influence of N_L but he has considered for Global (Secondary) deformation (see Figure 2.1 (c)) while Westergaard (1930) has considered mainly ***S*** incorporated in his analytical model (see Figure 2.1 (b) Local (Primary) deformation). According to the design viewpoint and extending back ground in parameters of Cao, dimensionless ratios of $\boldsymbol{S/S_{MIN}}$ and $\boldsymbol{L/L_{MIN}}$ have provided due to both effects of global and local deflections along location of girders for deck slab design.

2.2.2 Semi-Continuum Approach

The deck slab can be also analyzed by the semi-continuum idealization, which is represented by discrete longitudinal members and a continuous transverse medium as conducted by Bakht and Jaeger (1987). However, most actual bridge structures will distribute an applied load to an extent, which lies between the "no-torsion" grillage condition and the "full-torsion" of an isotropic slab (upper limit) behavior. This approach also studied by Rowe (1962) as it was referred to the load distribution by shear alone as "no-torsion grillage" which is not recognized in practical case. However, in realistic, some resistance to both bending and torsion should be presented in both longitudinal and transverse members as well as resistance to bending and shear at each joint between the longitudinal and transverse one.

2.2.3 Orthotropic Plate Theory

The orthotropic plate theory is used when the bridge superstructure is replaced with an equivalent plate having different elastic properties in two orthogonal directions. In the case of concrete deck slab replaced by thin plate, the slab is usually known as an orthotropic slab. Based on orthotropic method, there are numerous studies conducted in literature such as Troitsky (1967), Cusens et al. (1975), and Cao (1996), etc. In those researches, the deck was modeled by an orthogonal plate with flexural (θ) and torsional (α) rigidities.

Troitsky (1967) investigated the orthotropic plate that gives small load distribution to floor beam of bridge. The spacing of the main girder had a practically negligible influence on the stresses since loadings over its width were always placed symmetrically about the longitudinal center line of the bridge. Furthermore, convergence of this analysis will be happen when a smaller spacing between the main girders than the actual spacing is considered. Cusens et al. (1975) produced charts for orthotropic deck slabs and assumed that slab cross-section deflected in a smooth curve. Johnson et al. (1986) investigated an orthotropic plate equation for design of deck slab followed British Standard 5400 (BS 5400). They also stated about limitations of the orthotropic plate equation which is written as follow:

$$D_x \frac{\partial^4 w}{\partial x^4} + 2H \frac{\partial^4 w}{\partial x^2 \partial y^2} + D_y \frac{\partial^4 w}{\partial y^4} = p(x) \qquad (2.1)$$

where

D_x, D_y = Flexural rigidities

H = Torsional rigidity

They found that limitations of applicability are the maximum of 20° skew, simply-supported decks, a composite structure of beam and slab deck system with the minimum five equally spaced of uniform section beams, and no transverse members other than the concrete slab and diaphragms over the supports.

The 2-D mathematical model of Timoshenko and Woinowsky–Kreiger (1959) that was one of the best models for thin plate as follow:

$$\nabla^4 w = \frac{\partial^4 w}{\partial x^4} + 2\frac{\partial^4 w}{\partial x^2 \partial y^2} + \frac{\partial^4 w}{\partial y^4} = \frac{p(x)}{D} \tag{2.2}$$

where

w = the vertical translation

x = the transverse coordinate

y = the longitudinal coordinate

p = the vertical load

D = the plate rigidity (v is Poisson's ratio of the deck), equal to

$$D = \frac{Et^3}{12(1-\upsilon^2)} \tag{2.3}$$

In general, the above equation is based on several assumptions. For instance, the material should behave linearly elastic; the strain profile is linear; the plate is isotropic; the vertical stresses due to applied load are neglected; and the deformations are small relative to dimensions of the plate.

Although the orthotropic plate concept is relatively simple and offers practical advantages, this method appears to inaccurately predict the transverse moment in the bridge with few girders and wide as proved by Surana et al. (1998). In general, the orthotropic plate theory can reliably predict the static response but the transverse bending moments could not be accurately obtained if there were complex combination of bending between girders and bending due to non-uniform girder deflections. In practice, this method has been superseded by grillage analysis.

2.2.4 Grillage Analogy

In relation to the grillage method, the bridge is modeled by longitudinal grillage beam elements whose constants are usually calculated based on the composite girder-slab properties and by transverse beam elements based on the slab properties. Several researches on the bridge decks have been done based on this approach such as Bakht and Jaeger (1987), Hambly (1991), and Surana and Agrawal (1998). Generally, the grillage analogy gives conservative results for design propose and the most difficult part is the equivalent sectional property calculation of grillage member (Chan and Chan, 1999). The grillage analogy consists of skeletal members rigidly connected to the other at ***n*** nodes. In addition, it has ***3n*** degree of freedoms (DOFs) per node which are horizontal displacements and rotations about vertical axis are negligible in the analysis.

Detailed recommendations on the implementation of a grillage analysis for slab bridges can be found in Hambly (1991) and *etc*. Such simple models allow only for a global evaluation of bridge behavior. The accuracy of these calculations depends on the assumed location of the neutral axis in bending elements (O'Brien and Keogh, 1999). Many researchers started their modeling techniques by creating two dimensional grillage models to develop bridge construction practice. SAP 90 (1989) and SAP 2000 (2002) were used to create the two-dimensional models. By following the modeling methods of Heins and Firmage (1979), they used beam element for the longitudinal girder. Diaphragm member was transform into an equivalent plate and modeled as beam element. However, they noted that the grillage model could not capture the warping torsional stiffness exhibited by both the girders individually.

2.2.5 Finite Element Method (FEM)

In cases of unusual deck geometry, bridge designers may find it beneficial to employ refined methods of analysis. Typically the use of the refined methods of analysis is meant for the design of both of the girders and the deck slab. For any type of structure, the more complicated its structural geometric configuration is, the more a computer-based numerical solution becomes necessary. It has also been shown that experimental investigations are time consuming, capital intensive and even often impractical. The FEM is now firmly accepted as a very powerful general technique for the numerical solution of a variety of problems encountered in engineering. For concrete structures, because of complexities of concrete behaviour in tension and compression together with integrity of concrete and steel, extreme difficulties are encountered in modelling and obtaining closed form solutions, even for very simple problems. The engineering structures are today designed with respect to the limit state of serviceability and limit states of the strength and stability. These complex problems of a different nature are possible to solve by FEM methods. Nonlinear elastic concrete models have been extensively used in finite element analysis of reinforced concrete structures with varying degrees of success.

In FEM, the structure is idealized by continuum elements such as beam, plate, shell or solids elements. The different possible combinations of elements used in Finite Element have now improved. Subsequently, this approach becomes more and more popular in bridge deck analysis. In some cases, the slab is divided using shell elements and girders are represented using beam elements (Hays, et al., 1997). Diaphragms (if considered) are also represented by beam elements. In such plane models, centroid of beams coincides with the centroid of the slab.

To determine the cross-section properties of the beam, the actual distance between its neutral axis and the middle plane of the slab must be taken into account. Ebeido and Kennedy (1996) performed intensive finite element analysis on skew composite girder bridges. They use linear shell element with six degree of freedom at each node to model the concrete deck slab. Girders were modeled using three dimensional linear beam elements with also six DOFs at each node. These beam elements were also used to model diaphragm and cross frame bracing. Constraints were applied between the shell node of the concrete deck slab and the beam node of the longitudinal steel girders to ensure full interaction. Fang et al. (1990) performed testing on bridge deck slab designed with the empirical method. They used linear and quadratic thick shell elements with three DOFs at each nodes to model the slab and three dimensional beam elements with six DOFs at each nodes located at the girder mid-height. No slip was assumed between the slab and the girder. Despite the use of rigid link to connect space beam elements and shell elements, and to account for the eccentricity of the girders, it is still difficult with this method to include a precise composite action when determining beam stiffness.

Chung and Sotelino (2006) developed finite element models for composite steel bridge girder to improve the concrete material model. By using ABAQUS software program to create composite steel bridge system, they used three-node beam Timoshenko elements (B32 element) as the steel girder section connected to shear-flexible eight nodes shell elements (S8R elements) as the concrete deck. To assure the full composite action between girders and concrete deck, rigid links were used to connect from the centroid of the girder section to the centroid of concrete deck. Shell element was created using layer approached to more accurately represent concrete behaviors such as crack propagation, dowel action, and tension stiffening. To verify the modeling methods, the modeling results was compared to the results from experimental testing. The predicted and measured vertical deflections were very close, within nine percent difference.

Sometimes, the bridge behavior can be strongly affected by the structural components such as sidewalks, curbs, and barriers. In such cases, it can be incorrect to model them only by changing the thickness of shell elements. The application of solid elements also allows for a more detailed investigation of local stress and strain distribution. Modeling the slab with solid elements, and the girders and diaphragms with shell elements, seems to describe most adequately the bridge geometry and physical properties. However, using these high order element types always leads to expensive data input and time consuming in execution of the model. The evaluation of FEM models for bridges shows a tendency towards more complex model geometries with a larger number of elements. At the same time, the determination of element properties is clearer and stands closer to reality.

2.2.6 The Earlier Analytical Method of Westergaard

Based on the early study by Westergaard (1930), it was one of the significant analytical investigations on the effect of local area of a wheel load on a simply-supported deck moment ***(M_{ox})***. He published and proposed a method of computation of stresses of wheel pressure over a small area due to a concentrated load (***P***), which was assumed to be distributed uniformly over a circle of diameter "*c*", and the Poisson's ratio was *0.15*. The initial assumption was that the slab extends sufficiently far in the direction ± *y*, and considering as an infinite slab without being supported by diaphragms. He suggested an alternative method for deducing ***M_{ox}*** which is based upon the effective width of a simple beam (***B_E***) and has an applied load ***P*** at mid-span. He also showed that his method obtained almost the same result as of the exact method, and was much simplifier than others one. The model proposed by Westergaard (1930) is shown in Figure 2.2.

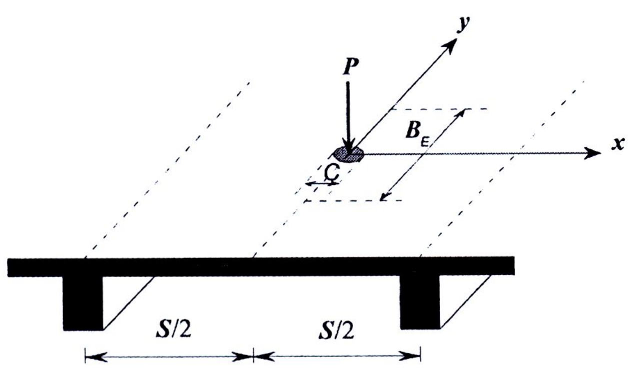


Figure 2.2 Effective width of the slab (***B_E***) and infinitesimal region (*c*) of a load ***P***

An initial computation of Westergaard (1930) gave only the stresses in bridge slabs due to local action of wheel loads from condition of the continuity slab on rigid vertical supports without considering its torsional restraint. This method was based on the slab analysis of infinite length. However, Rowe (1962) pointed out that such method was valid only when a spacing of diaphragm is greater than three times of the main girder spacing. Furthermore, Westergaard developed the formula to compute the maximum moment per unit width of slab by assuming that the moment was distributed uniformly over a certain width of slab of simple span which is termed the "effective width" in the direction of perpendicular to the span of the slabs. The formula was shown below.

$$M_{ox} = \frac{PS}{4B_E} \tag{2.4}$$

where

M_{ox} = the maximum transverse moment per unit width of slab

P = the load applied at the center of the bridge span

S = the spacing of the slab support (center-center beam spacing) in foot

B_E = effective width of the slab in single load at center of span

in which $B_E = 0.58S + 2c$, ft.; c = the diameter of the wheel load assumed equal to 1.25 ft. The maximum moment formula can be then rewritten as below.

$$M_{ox} = \frac{PS}{2.32S + 8c} \tag{2.5}$$

Notably, the above formula proposed by Westergaard (1930) was adopted by AASHTO Standards in 1935 for computing the maximum moment per unit width of slab. However, to calculate bending stresses due to wheel load on floor slabs in this solution, it was based on assumption of no distribution in the direction of the span of the slabs. Later, although the analysis of the influence surfaces of the deflection of plates was done by Westergaard, the obtained results did not use to develop the above equation.

2.2.7 Simplified Analytical Method of Cao

A more aggressive approach to mitigating steel corrosion is to completely remove all steel from the top mat as proposed by Cao (1996). The slab theory of Westergaard (1930) requires the placement of reinforcing steel near the top of the deck to resist negative moments that occur over the girders. However, Cao (1996) concluded that slab theory alone was not sufficient for use in deck design. Due to the underlying differential deflections between girders, the negative moments incurred in the deck slab were less than those predicted by a slab supported on rigid girders. Cao (1996) performed live load testing on the South Platte River Bridge near Commerce City, Colorado to evaluate this conclusion. Results showed that the top mat of reinforcing steel was not necessary to withstand the negative bending moment over the girders induced by truck loads. Although the conclusions of Cao (1996) have not been included in the bridge design provisions of AASHTO, Cao's findings have important implications for properly interpreting live load testing results and should be taken into consideration.

Cao (1996) studied analysis and design of slab-on-beam of highway bridge decks without cracking consideration. He evaluated the positive bending moment in a deck slab induced by the differential deflections of beam using moment-displacement relation of a two-edge-simply-supported slab subjected to a wheel load. Moreover, the differential deflection between beams in a deck subjected to a single axle load by the orthotropic plate theory, the beam stiffness, spacing and span length of the bridge beam effects to the maximum negative bending moment in a deck slab could be assessed. His simplified analysis method was evaluated with a numerical parametric study of the deck on three girders and a linearly elastic finite element. Then, the deck slab was modeled with 4-node plate elements, and the girders were modeled with 3-D beam elements which were connected to the bottom of the slab with stiff beams. The elastic rigidities of an orthotropic plate were estimated with a deck section similar to a T-beam.

To account for influence of the flexibility of the supporting girders, Cao (1996) has proposed the following formula to compute the maximum negative moment in the slab-on-girder deck.

$$M_t = M_l + K_d M_o \tag{2.6}$$

in which M_l is the primary moment as illustrated in Figure 2.1(c), which is the negative moment in a slab if it were supported on rigid, and $K_d M_o$ represents the secondary moment introduced by the girder deflection as shown in Figure 2.1(b). With the assumption that the wheel load P is placed at the mid-span, Cao has derived an equation based on the plate theory to calculate the primary moment M_l. He has further shown that M_l can be accurately estimated with the simple expression of -0.2146$\boldsymbol{P}$ for the cases with $\boldsymbol{S/L} < 0.75$, where $\boldsymbol{S}$ is the center-to-center spacing of the girders, and $\boldsymbol{L}$ is the span length of the bridge deck. The above condition is valid for most bridge decks. M_o is the positive moment in a two-edge simply supported infinitely wide slab to be calculated by Equation (2.7), which is proposed by Westergaard (1930) as follows:

$$M_o = \left\{ \left[(1+\nu_C) \ln \frac{4S}{\pi c} + 1 \right] / 4\pi \right\} P \tag{2.7}$$

in which $\boldsymbol{c}$ is the diameter of the equivalent wheel load and v_c is Poisson's ratio of the deck concrete. Cao has assumed $\boldsymbol{c}$ to be 380 mm (1.25 ft) for a 71.2 kN (16 kips) wheel load according to the tire area specified in the AASHTO Standard Specifications (1992). It should be noted in Equation (4.7) that the solution for M_o in a slab, which is simply supported at two opposite edges and subjected to a wheel load was given by Westergaard (1930) only. The positive bending moment in a deck will increase with girder deflections. However, this increment is not considered in the AASHTO specifications. For the sake of simplicity, it is reasonable to assume that the maximum positive bending moment in a three-girder deck supported on rigid girders occurs when a wheel load is applied at the middle of each span, as shown in Figure 2.3. Note that the effects of numbers of deck supported and lanes loaded is not significant.

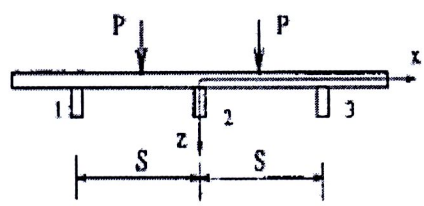


Figure 2.3 The maximum positive bending moments induced by differential 3-girder deflections

The factor K_d is calculated with the Equation (2.8) below, which is derived by Cao (1996) based on an orthotropic plate theory to account for the different bending stiffness of a deck in the longitudinal (D_y, parallel to the traffic) and transverse directions (D_x).

$$K_d = K_{do} \sin\frac{\pi y}{L} \tag{2.8}$$

where d is the distance of a wheel load from the nearest support of the deck;

$$K_{do} = 0.2176\left(\frac{L}{S}\right)^2 \sqrt{\frac{D_x}{D_y}} e^{-\frac{\pi\alpha S}{2L}} \left[\left(\frac{1}{\alpha}\cos\frac{\pi\beta S}{2L} + \frac{1}{\beta}\sin\frac{\pi\beta S}{2L}\right) - \left(\frac{1}{\alpha}\cos\frac{3\pi\beta S}{2L} + \frac{1}{\beta}\sin\frac{3\pi\beta S}{2L}\right) e^{-\frac{\pi\alpha S}{L}}\right] \tag{2.9}$$

In which the elastic rigidities of a deck (D_x and D_y in the transverse and longitudinal directions, respectively), α, β and I_{gc} can be calculated as follows:

$$D_x = \frac{E_c t^3}{12(1-\nu_c^2)}; D_y = \frac{E_g I_{gc}}{S} \tag{2.10}$$

$$\alpha = \left[\frac{1}{2}\sqrt{\frac{D_y}{D_x}} + 1\right]^{1/2}; \beta = \left[\frac{1}{2}\sqrt{\frac{D_y}{D_x}} - 1\right]^{1/2} \tag{2.11}$$

$$I_{gc} = I_S + I_g + A_g e_1^2 \tag{2.12}$$

$$I_S = \frac{E_c}{E_g(1-\nu_c^2)}\left(\frac{St^3}{12} + Ste_2^2\right) \tag{2.13}$$

where a T-section in a deck whose flange width is equal to ***S***; e_2 = distance between the neutral axis of the composite section (T-shape in Figure 2.4) and the mid-plane of the slab; e_1 = distance between the neutral axis of the T-section and that of the girder; E_c is the modulus of elasticity of the concrete deck; E_g is the modulus of elasticity of the girder; A_g and I_g are the area and moment of inertia of the girder; I_S is the moment of inertia of the slab section; I_{gc} is the moment of inertia of the T-section.

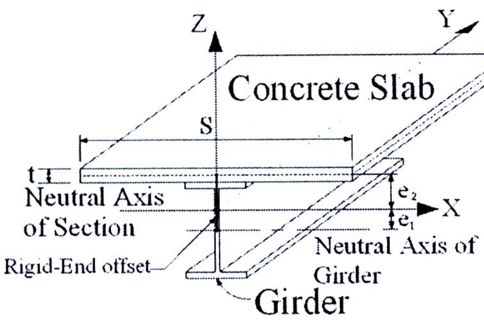


Figure 2.4 3D physical T-Section of a slab-on-girder bridge

Equation (2.8) indicates that K_d approaches zero as ***y/L*** approaches zero, and assumes the maximum value at ***y/L*** = 0.50 (mid-span), where the girder deflection is at the maximum. From the above equation, one can see that the influence of the girder deflection on the slab moment depends on D_x/D_y, ***S/L***, and the distance ***y/L***. Cao has verified this analytical method with the experiment results obtained by Fang et al. (1990) and also by finite element models (SAP 90).

At this time, it has been observed from the test and simplified analysis of Cao (1996) that the differential deflection between girders in a slab-on-girder bridge reduces the negative bending moments in the deck slab. Cao considered the effect of girder deflection based on an elastic plate theory of 3-girder deflections and also validated with finite-element models using computer program SAP 90 (1989). With this approach, the effects of number of lanes loaded on the maximum negative slab moments, however, cannot be assessed.

2.3 Bridge Deck Design by Traditional Codes of Practice

2.3.1 OHBDC and CHBDC Design Codes

In 1983, Canada originally developed a deck design by taking advantage of the arching behavior, commonly referred to as the 'Ontario' deck design. OHBDC (1983) recommended the new design of slab-on-girder bridges according to an empirical method, which conforms to certain conditions by using at least 30 MPa of a slab concrete strength, at least 1.00 m (3.3 ft) slab overhang beyond exterior girders, lesser slab span of 3.70 m (12 ft), the maximum 0.30 m (12 in) spacing of reinforcing bars, 8.00 m maximum spacing of diaphragms, the presence of diaphragms at all supports and so on.

More recent Canadian Codes (OHBDC 1991, CHBDC 2006) have taken this most aggressive approach to steel reduction by designing steel free decks, which take advantage of arching action to completely eliminate all of the reinforcing steel within the deck (Bakht and Lam 2000, *etc*). The arching action is most effective in slabs with 0.3% of a minimum reinforcement for the top of the deck. To indicate increased strength, the suggesting membrane action is possibly contributing to load carrying capacity by performing live load moving in a concrete slab. Methods of construction is possible, and is still quite common, to cast the concrete slab.

In lieu of the internal steel reinforcement, exterior steel 'straps' are connected between adjacent girders to provide the lateral confinement necessary to maintain arching action. Other 'steel-free' deck designs utilize fiber-reinforced polymer (FRP) bars or grids within the deck concrete in place of traditional steel rebar. Both steel-free methods effectively mitigate any damage due to corrosion by complete removal of the corrosive medium (i.e. reinforcing steel), regardless of concrete integrity. In order to fulfill certain geometric criteria of bridge decks, the Ontario approach allows the total amount of reinforcing steel to be reduced by approximately 30%. The primary design guideline requires a minimum steel area of 0.3% of the concrete area in both the top and bottom reinforcing mats to help control cracking and maintain confinement within the deck. The typical cross section of such steel-free deck is illustrated in Figure 2.5.

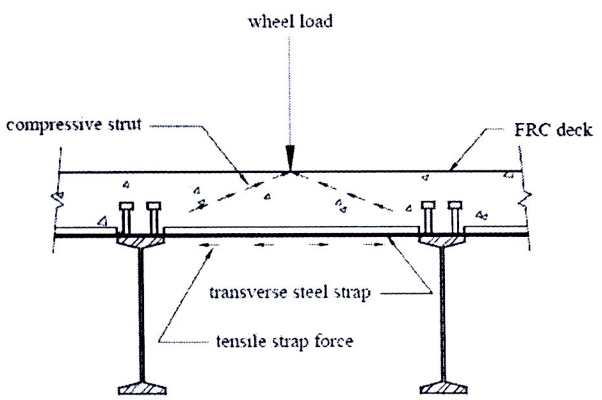


Figure 2.5 Cross section of a typical steel free deck

A number of tests confirmed that a proper design of a steel free deck (SFD) system does not fail in a flexural mode but a punching shear mode when external point loads were applied between the supporting girders. These failure loads were up to four times higher than the resistance required by the Ontario Highway Bridge Design Code (OHBDC). The Canadian Highway Bridge Design Code (CHBDC), which replaced the OHBDC in December of 2000, includes a design section for SFD system consisting of a composite fiber reinforced concrete (FRC) deck that does not contain any internal reinforcing steel.

2.3.2 AASHTO Standard and LRFD Specifications

2.3.2.1 Live Load Considerations for Highway Bridge Design

Concrete deck slabs on bridges designed in accordance with AASHTO Standard Specifications for Highway Bridges using Allowable Stress Design (ASD) are to be designed under a standard HS20-44 truck loading. For any highway bridge design, the standard 196 kN (20-Ton) truck with trailer combination is signified by HS20-44 truck and spaced 1.2 m (4 ft) apart is used as a minimum distance. The lane load occupies a 3 m (10 ft) width and transverse distance of centerline of wheel load equal 1.83 m (6 ft). For design, "Lane" loading is used instead of the actual vehicle loading if these results give a higher bending moment. This "Lane" loading is the same for HS 20-44 loading of the same weight and amount for the HS loading to a uniformly distributed load of 9.34 kN (0.952 Ton)/m (0.64 kpf), and a concentrated load for bending of 80 kN (8.165 Ton) per 3 m (18 kips per 10 ft). The longitudinal distances between wheel loads of HS20-44 truck are shown in Figure 2.6.

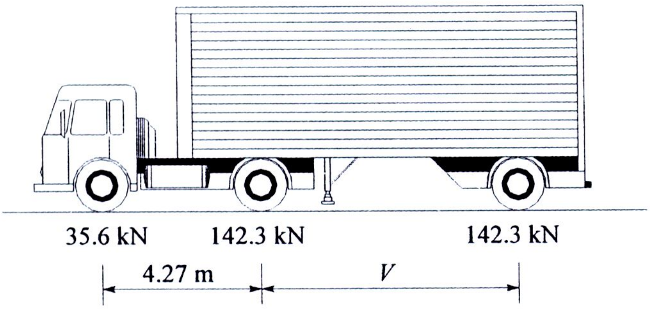


Figure 2.6 Standard HS 20-44 truck according to AASHTO Specifications

where V represents the variable spacing of 4.27 m to 9.14 m (14 ft to 30 ft), which produces maximum stresses

2.3.2.2 AASHTO Standard Specifications

The current formulas for slab design moments due to wheel loads, Article 3.24.3.1 in the AASHTO Standard (2002), first appeared in the 8th edition of the AASHTO

Standard (1961) in 1961. Prior to that, a slab distribution width was used for determining the slab design moments considering both a single axle and tandem axle according to the 7th edition of the AASHTO Standard (1957). In all cases, a continuity factor of ***0.8*** was to be applied to positive and negative moments for deck slabs continuous over three or more supports. Tire contact area was added to the AASHTO Standard, 12th Edition (1977) was intended to allow for more accurate bridge deck analysis. The tire contact area was to be assumed as a rectangle with an area in square inches of $0.01\boldsymbol{P}$, in which $\boldsymbol{P}$ is the wheel load in pounds. Equating this area for a 16 kips wheel load to a circle results in a diameter of 1.19 ft, which is almost similar to the 1.25 ft used by Erps et al. (1937).

For the evaluation of the positive and negative moments in a deck slab, the AASHTO Standard Specifications (AASHTO 2002) adopts a simple approach by specifying both moments be taken as 80 percents of the maximum moment in a simply supported slab subjected to the design wheel load. The 80 percents is a conservative estimation of the continuity factor for a continuous beam on rigid supports. The maximum moment per unit width in a simply supported slab is calculated. For the 1st edition in 1931 AASHTO, the design philosophy utilized in the Standard Specifications was Working Stress Design (WSD) approach. It should be noted that the AASHTO Standard Specifications (AASHTO, 2002) is originated from a well-known research by Westergaard (1930).

2.3.2.3 AASHTO LRFD Specifications

In 1993, AASHTO adopted the "Load and Resistance Factor Design of Bridge Design Specifications" (LRFD specifications) as an alternative to the "Standard Specifications for Highway Bridges" (standard specifications). Its adoption raises many questions regarding the specification's impact on the resultant bridge members' proportions and the design process itself. The implication of the provisions of the LRFD specifications on the design of steel highway bridges relative to those of the load factor design (LFD) provisions of the standard specifications is investigated through a dissection of the specifications into the load and resistance sides of the LRFD equation. A simple design example illustrates the impact of the LRFD specifications. Finally, the design process and effort required to apply each set of provisions, LRFD and LFD, are discussed on the basis of the example.

AASHTO LFRD Bridge Design Specifications (1994) permitted an empirical slab design for only composite decks with deck slenderness ratio between 6 and 18. The adoption of recommending was for an isotropic steel arrangement with a minimum steel ratio of 0.3% and 0.2% for the bottom and top of the deck slab, respectively. The last revising 17^{th} edition (Standard 2002) was from an empirical method, and it was predominantly based on the WSD philosophy. The AASHTO Specifications (LRFD and WSD) also suggest performing a more exact analysis by refined methods such as FEA, an orthotropic plate model, etc., while considering a footprint of a tire contact area. The critical moment in any refined method has to multiply by the dynamic load allowances (*IM*) and the multiple presence factors *m* to account for the impact effect and the unlikeness to load heavy trucks simultaneously in adjacent lanes, respectively. The factor *m* of LRFD is equal to ***1.20*** for one loaded lane and the other ones are less than 1.00 for three loaded lanes in rare situations.

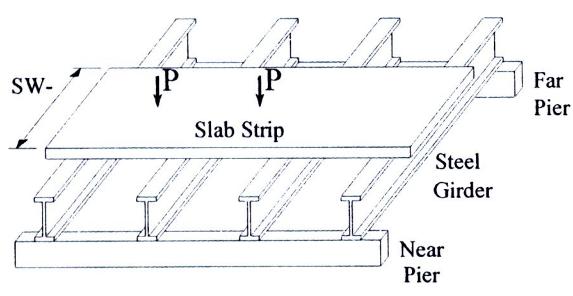
Nowadays, the AASHTO LRFD Code permits three methods or procedures for designing bridge deck slabs with primary reinforcement perpendicular to the main bridge beams. As outlined in Article 9.6.1, 3 methods are: (a) Approximate Elastic or "Strip" Method (Article 4.6.2.1) or Traditional Design Method; (b) Empirical Design (Article 9.7.2); and (c) Refined Analysis (Article 4.6.3.2). Based on component testing and finite element analysis (FEA), AASHTO (2000) has included provisions for the 'empirical design', which is similar to the Ontario deck design, except that a further reduction in the top layer of reinforcing steel is allowable, requiring a minimum top-mat steel area of ***0.2%*** of the concrete area. A minimum bottom-mat steel area of ***0.3%*** of the concrete area is required for improving crack control and to maintain confinement within the deck.

The last 3^{rd} edition of LRFD (AASHTO, 2004) provided both table and formula, which offers an equal alternative to WSD (AASHTO Standard, 2002). Following to this code, the slab thickness (*t*) also should be one-fifteenth of deck slab span with a minimum of 225 mm, and the reinforced concrete slab should comprise two orthogonal meshes with a minimum concrete area of a steel ratio of ***0.3%*** in the transverse and longitudinal directions for each steel layer.

The Empirical Design Method relies on the arching action of concrete between the supports shown in the tabulated moment values M_{TAB}, AASHTO (Specifications 2004) recommends (see Article 4.6.2.1.6) that the negative design moment is taken at a distance of ¼ and ⅓ of the top flange width from the centerline (C_L) of the steel and PC-girder, respectively whereas the maximum positive design moment is taken under a wheel load. Not only M_{TAB} can be taken, but a computable analytical moment ($\boldsymbol{M_{ANA}}$) is another alternative design or second method called the "Traditional Design Method". It is typically referred to as the equivalent strip method. It is based on the approximate elastic method specified in Article 4.6.2.1 and is consistent with the AASHTO's Standard Specification. This method, based on Westergaard's formulas (Westergaard, 1930), has been used successfully in the past by Kansas Department of Transportation (KDOT).

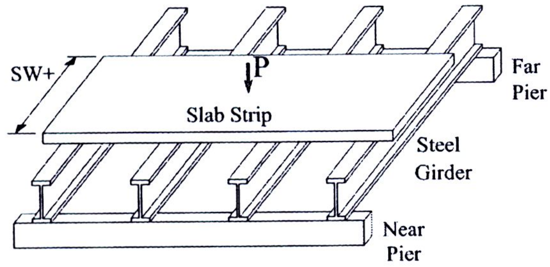
The equivalent strip method (transverse reinforcement) is based on the following: (1) A transverse strip of the deck is assumed to support the design loads and is treated as a continuous beam; (2) The strip is assumed to be supported on rigid supports at the center of the girders which do not deflect; (3) The loads are moved laterally to produce moment envelopes, and multiple presence factors together with dynamic load allowance are included; (4) Factored design moments are then determined using the appropriate load factors for different limit states; (5) The reinforcement is designed to resist the applied loads using conventional principals of reinforced concrete design; (6) Single reinforced cross section should be used in analysis; (7) Shear and fatigue of the reinforcement need not be investigated; (8) Concrete slabs will have four layers of reinforcement, two in each direction, that comply with Article 9.7.3.1.

In lieu of the tabulated values, one can also compute the moments recommended in Table 4.6.2.1.3-1 of section 4 of the Specifications. This is called the analytical strip approach in this study. When using the recommended Equivalent Strip Width (*SW*), the slab moments *(M_{ANA})* can be computed with an equivalent one-way bending strip as illustrated in Figure 2.7. To this end, the engineer needs to conduct two separate analyses for the M^-_{LL} and M^+_{LL} with the respective *SW*. For cast-in-place decks, the effective strip widths for negative bending (***SW***$^-$) and positive bending (***SW***$^+$) are equal to 1220+0.25***S*** mm (48+3***S*** in) and 660+0.55***S*** mm (26+6.6*S* in), respectively. Through this approach, the moments are computed by ignoring the deflection of the girders.



(a) For ***SW***$^-$

Figure 2.7 Effective Strip Width (***SW***) of concrete deck supported on parallel girders



(b) For SW^+

Figure 2.7 (Cont't) Effective Strip Width (***SW***) of concrete deck supported on parallel girders

The developed procedure of Strip or Approximate Elastic Method is very similar to the slab design procedure in AASHTO Load Factor Design (LFD) and thus provides a measure of continuity for engineers during the transition process from LFD to LRFD. Refined Analysis utilizes finite elements, which is unnecessary for standard deck design. Empirical Design employs the notion that the deck behaves more like a "membrane" than a series of continuous beams. While this may be true, it is not a well enough established design technique to be advocated by Illinois Department of Transportation (IDOT).

In the Approximate Elastic Method, the deck is designed for Flexural Resistance and Control of Cracking. Limits of reinforcement are also checked, but do not typically control in a standard deck design. Shear design is not required for deck slabs. Fatigue and fracture design is also not required. In a standard deck, three components are designed. Positive moment (bottom of slab transverse) reinforcement and negative moment (top of slab transverse) reinforcement are designed for the Approximate Elastic Method. Additional negative moment reinforcement for deck overhangs should also be designed with a significant crash loading normally governing.

2.3.2.4 Design Procedure in AASHTO (Main reinforcement perpendicular to traffic)

There are some important recommendations of bridge specifications as follows:

AASHTO Standard Specification for Highway Bridges or AASHTO WSD (Standard 1957) adjusted Westergaard's original formula to obtain the maximum bending moment per one foot width of slab as follows:

$$M = \frac{PS}{4(0.6S + 2.5)}, 2 \le S \le 7\,feet \tag{2.14}$$

$$M = \frac{PS}{4(0.4S + 3.75)}, S > 7\,feet \tag{2.15}$$

AASHTO WSD (2002) noted about having more exact methods of computing stresses in floor slabs due to concentrated loads that might be found in "Public Roads" for March, 1930, in an article by Dr. H. M. Westergaard entitled "Computation of Stresses in Bridge Slabs due to Wheel Loads". Moreover, AASHTO specifications which based on elastic analysis and suggested the bending moment per one foot width of slab which main reinforcement perpendicular to traffic shall be calculated according to method given under cases A unless more exact method considering the tire contact area in article 3.24.3.1.

The specification suggestion of load distribution factor for concrete slabs on beam bridge according to Case A-Main Reinforcement Perpendicular to Traffic (Spans 0.61 to 7.32 m inclusive) will be the main attention of this study. In addition, the AASHTO specifications recommend that in slabs continuous over three or more supports, a continuity factor of ***0.8*** shall be applied to the simple span live load moment for both positive and negative moments. However, at service load design, the AASHTO specifications do not involve the elastic behavior of the continuous deck slab on elastic supports (support stiffness) to the bending moments of the slabs. In fact, the settlement of supports directly affected the bending moments in slab. Following the empirical formula in 1961, AASHTO has proposed the main transverse moment of deck slab over three or more steel I-girders until 2002 (excluded impact) as follows:

For HS 20 Loading:

$$\text{Moment in foot-pounds per foot-width of slab} = \frac{(S+2)P_{20}}{32} \tag{2.16}$$

$$\text{Moment in Ton-meter per meter-width of slab} = \frac{(S+0.610)P_{20}}{9.754} \tag{2.17}$$

The concrete slab bridge is always supported by at least three girders; consequently, the Codes have recommended a continuity factor of ***0.80*** for multiplying to the mid-span moment of a simply-supported deck to obtain both moments of the continuous slab. Therefore, the top transverse reinforcing bar will be used in the same amount as the bottom one. Obviously, the continuity factor ***0.80*** is base on one single deck span-on-rigid simply-supported. The Codes have also recommended the location where negative slab moments should be used for the design as follows:

- For steel I-beams: one-quarter the flange width from the centerline of support shall be considered.
- For precast I-shaped concrete beams: one-third the flange width, but not exceeding **15.0** in, from centerline of support shall be considered.

AASHTO LRFD Specifications is used for design procedures now. For concrete slabs supported on steel or prestressed concrete beams or PCI girders, the live load moments shall be computed in accordance with Article 4.6.2. Moments in Article Table A4.1-1 can be used directly if the assumptions and limitations listed at the beginning of the appendix are fully met. This table is used with the center to center girder spacing S for positive moments and the design location from the centerline of the girder for negative moment. Dynamic allowance and multiple presence factors ***IM*** are already included in this table. Skew effects are to be considered per Article 9.7.1.3. In general, the effectiveness of the reinforcement is reduced as the skew increases. KDOT allows the maximum rebar placed parallel to the skew to deal with this difficulty. For construction purposes KDOT allows reinforcement to be placed in the skew up to ***35°***. The minimum reinforcing bar clearance for top bars must meet the requirements for bridge deck protection as mentioned in Article 3.9.2 Protection for Bridge Decks. For the bottom reinforcement, the bar clearance shall be 1½ in. This is a change from the past KDOT policy because the maximum aggregate size in the future will be 1 in.

2.4 Bridge Deck Characteristics for Optimum Design

2.4.1 Practical Reinforced Concrete Material of Superstructure

Simplified material models intended to be employed in the analysis of concrete structures. Mostly making use of the assumption of material property is elastic. Poisson's ratio did not show significant effect (Wegmuller and Amer, 1975). The 28 days concrete compressive strength of 20.1 MPa (3000 psi or 210 ksc) to 27.56 MPa (4000 psi or 281 ksc) is usually satisfactory for most requirement of the structure, and slightly higher strength might be preferred if greater durability was desired. For the girder of medium-span prestressed concrete bridge, strength up to 41 MPa (6000 psi or 420 ksc) was commonly used (Heins and Firmage, 1979). Iles (2001) suggested types of simple span of suitable steel I-girder configurations such as universal rolled I-sections that are available up to 1016 mm (40 in) deep, but the section above 762 mm (30 in) serial sizes can be more economical by replacing with similar plate sections. In addition, precast post-tensioned I-girders have a useful span range of 20-35 m whereas precast post-tensioned T-girders are suitable for span ranges up to 45 m (Liebenberg, 1992).

2.4.2 The Physical Geometries

The design of a composite bridge deck consists primarily of the selection of the slab depth, spacing and section of the girder. The possibility of a large number of combinations of concrete slab thickness and number of girders makes the design process a trial and error procedure. Although numerous works have been done on the design of composite bridge deck, the selection of the cross section for optimum cost has paid relatively less attention. One of such is in a book by O'Connor (1971). However, the assumptions stated by O'Connor on which the formulation is based only on the depth of the girder can therefore not be regarded as a complete optimization for the composite system.

The deck slab has to distribute wheel loads to the main girders and also to transfer some load from more highly loaded girders to adjacent ones. The spacing of main girders (S) thus affects the design of the slab as well as the number of girders (N_G) required. For closely spaced girders, wheel loads determine the design of the slab including its reinforcement. On the basis of typical shear and crack width requirements, the minimum thickness of slab is about 220 mm (8.66 in). The total transverse moments in the slab are not very sensitive to girder spacing in the range 2 m (6.56 ft) to 3 m (9.84 ft) since the increase in local moments as the spacing increases is almost balanced by the reduction in moments from the transfer of load between girders. The optimum slab thickness t is typically 200 to 250 mm.

In comparison with transverse section, N_G will be represented a width of slab from left-to-right exterior girders, which is obtained by multiplying the girders spacing S with N_G-1. For the conservative purpose, the standard lane width is taken as 3 m (10 ft). The number of girders N_G is varied from 3 to 7 and equal to 3 for negative and positive slab moments, respectively. To decide the width of highway bridges, the first step is to define the number of notional lanes on the bridge. One-, two- and three-lane deck-on-girder are evaluated for the critical negative moments only when the minimum numbers of longitudinal girders are three, four and five, respectively. Not like the simplicity of the positive slab moment, it is needed to assume the different patterns of load lanes on bridge deck.

2.4.3 Physical Arrangement of Girders

The most economical for arrangement of main girders of a slab-on-girder highway bridge can be determined by the construction experience. Furthermore, many researchers have studied effects of some major parameters of composite girder bridges on load-deflection behavior. Those considered parameters included beam size, effect of presence and number of cross diaphragms, torsional constant, slab thickness, Poisson's ratio, yield stress of steel girders and the ratio of transverse to longitudinal stiffness of the slab. In general, wide-flange beams (I-girder), which comprise nearly 50% of the tonnage of structural steel shapes rolled today are widely used. Accordingly, all I-girder sections used in the present study are referred from the 2002 edition of AASHTO LRFD Manual of Steel Construction in both U.S. customary units and metric units. For example, the W36x150 wide-flange girder is approximately 0.914 m (36 in) deep and weighing 2.19 kN/m (150 lb/ft).

A thicker slab distributes the load more evenly and has a uniform deflection response result. Reinforced concrete slabs are used for widely spaced girders and a wide range of composite bridges. Where they are supported at close centers, i.e. up to 3.5 m (11.5 ft), they will usually have a uniform depth of 220 to 250 mm. In such cases, the economies that can be achieved by reducing dead weight justify the extra cost of using variable depth slabs. In selecting a suitable girder spacing *S*, it is important to ensure that the cantilevers at the edges of the slab are limited to avoid overstressing the slab or forcing excessive load onto the outer girders. From the preceding discussion, the optimum proportions basis of experience are emerge: girder spacing as wide as is necessary and not wider than about 12 m (39 ft), cantilevers not more than 1.5 m (5 ft) if they carry traffic or about 2.5 m if they carry footways that are protected by crash barriers to avoid local wheel loading, spans up to about 35 m (115 ft) for the common economic composite bridges.

The diaphragm had become less popular since simplicity of construction was ever more important (Liebenberg 1992). Such estimates of girder size then permit better estimates of dead load of the structure to be calculated. Additional guidance on proportioning plate girders is given in many sources for highway bridges, common proportions for main girders are: depth (h) from $L_o/18$ to $L_o/12$, flange width (b) from $0.25h$ to $0.35h$, web thickness, t_w : t_w » $h/125$, flange thickness, t_f: $b/25$ to $b/10$, span-to-depth ratios, ***L/h*** : 15 to 18.

2.5 Code Recommendation in Details of Reinforcement

In this study, the superstructure characteristic will be based on the economy of construction such as using the minimum number of main girder, uniform thickness of deck slab, minimum thickness of deck slab due to wheel loads of AASHTO HS20 loading, and economical section of a simple girder in the short-span range. Moreover, the ratio of transverse to longitudinal bending stiffness of the slab and Poisson's ratio affect slightly deflection response but may be more for slab bending moments and in-plane forces. Deck slenderness of concrete-slab-on-steel-girder type depends not only on girder spacing, but also on flexural rigidity (***EI***) of flexible supports and appropriate boundary conditions have to be effect of concern.

In the United State, the basic manual of highway bridges has been designed according to AASHTO Specifications, and these specifications permit the use of either Allowable Stress Design (ASD) or Load and Resistance Factor Design (LRFD). Idealization of bridge deck in AASHTO Specifications is for live load moment per unit width of deck. Nowadays, obviously AASHTO empirical formulas (AASHTO Standard, 2002) for slab moments still require only the spacing of girder (***S***) which is based on simple rigid girders subjected to wheel load, and the excluding many influence parameters in the formula that affect to the interaction between slab and girders such as composite action, diaphragm effects, etc. The AASHTO Code gave a basically formula in flexure slab as a beam. Nevertheless, its behavior is not like a beam since longitudinal cracking seldom happen. In addition, the weakness plane is cracked often above main reinforcing direction. Note that the fact which most deck slab cracking is transverse, then the primary reinforcement places closest to the deck surfaces in order to have more effective in resisting the cracking. Furthermore, to modify the slab formula for continuity over girders, a continuity factor of ***0.8*** for both dead and live load moments is used in AASHTO formula.

Since traditional cast-in-situ concrete slab construction is economically superior to the use of precast unit in cost comparisons and speed of construction. Thus, mainly slabs on girder of highway bridges in Thailand are cast in place and usually familiar to designers by following requirement of the traditional method (AASHTO, 2002) with some limitation of geometry and boundary conditions. Moreover, there are preferred to place the simple end of girders on neoprene bearings (yielding supports) because it has practical properties such as: low cost, easy to be replaced, and require less maintenance.

Figure 2.8 shows the deck slab reinforcement details for three different girder systems. In general, the deck slab thickness and reinforcement schedule should be considered as a lower bound for a given span length. In addition to AASHTO LRFD (AASHTO, 2004), the following factors have also been considered in developing the deck design charts:

- Deck service life: In general, reinforced concrete bridge decks have been observed to have a satisfactory service life of at least 40 years. This service life has been extended through maintenance repairs.
- Fatigue: One of the factors that affects bridge deck life is its capacity under fatigue loading. Research has shown that the deck thickness has a significant

influence on its fatigue life. Hence, any reduction in deck thickness from that used in the past may adversely affect deck fatigue life.

- Deck flexibility: Thinner decks are more likely to load related cracking than thicker decks over the same girder spacing.

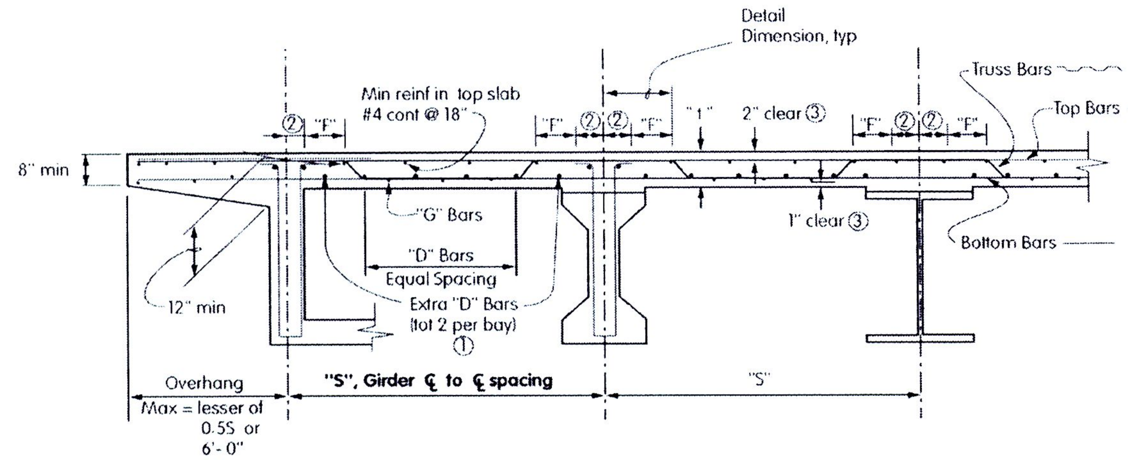


Figure 2.8 Deck slab reinforcement details

In AASHTO LRFD Code (AASHTO, 2004), the 1 percent minimum longitudinal steel requirement in the negative moment region is primarily to control transverse cracking of the slab. The area to consider for the 1 percent requirement should be the effective deck width multiplied by the thickness. This thickness is the total deck thickness, including the overlay, less an allowance for wear. In the past the dead load counter flexure was used to terminate a portion of the negative moment slab steel. The current AASHTO LRFD requirement in Article 6.10.1.7 states that *1%* of total cross-sectional area is the amount of reinforcement which must extend to a point where either (1) the factored construction loads or (2) load combination service II in Table 3.4.1-1 does not exceed ϕf_r, where $\phi = 0.9$ and f_r shall be taken as the modulus of rupture of the concrete determined as $f_r = 0.97\sqrt{f'_c}$. These stresses are calculated according to Article 6.10.1.1.1d. This reinforcement will have a yield strength not less than 400 MPa (60 ksi) and a bar size not exceeding 20-mm diameter (no. #6) with bar spacing not greater than 300 mm (12 in). The reinforcement is to be placed in two layers uniformly distributed across the deck width and *67%* placed in the top layer.