

ACI 342R-16

Report on Flexural Live Load Distribution Methods for Evaluating Existing Bridges

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Report on Flexural Live Load Distribution Methods for Evaluating Existing Bridges

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Report on Flexural Live Load Distribution Methods for Evaluating Existing Bridges

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This report provides a synthesis of the topic of flexural live load distribution and its applicability to concrete bridges. Flexural live load distribution is critical to describing how loads are transmitted through a bridge system. This report is intended to provide engineers, including load rating engineers, with basic guidance on the methods and tools available for determining live load distribution behavior of in-service bridges. Included in the report are descriptions, a brief history, and background of the flexural load distribution phenomena and a summary of design and analysis methods used to describe the phenomena in practice. A series of case studies are presented in the latter part of the report to serve as a comparison summary of commonly used live load distribution methods and their performance in describing the behavior of in-service structures. The report also provides performing bridge load ratings with a practical synopsis of the various methods available for determining the live load distribution factor. While this report is limited to flexural live load distribution, it provides the foundation for a future committee guide on the in-service evaluation of concrete bridges.

Keywords: bridge analysis; bridge load rating; distribution factor; equivalent beam analysis; finite element; grillage analysis; live load testing; load resistance; transverse flexural load distribution.

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CHAPTER 1—INTRODUCTION AND SCOPE

1.1—Introduction

Maintenance of an aging transportation infrastructure, including concrete bridges, is essential to the sustainability of resources and economic prosperity. With a national inventory of more than 600,000 bridges in the United States, 66 percent of which are concrete, maintenance and preservation represent a challenge for transportation agencies (Federal Highway Administration 2014). For these agencies, the challenge is to avoid or minimize bridge replacement and rehabilitation in the face of increased traffic volume and truck loads, along with dwindling financial resources.

Transportation agencies are responsible for ensuring both the safety and functionality of these bridges, and meeting this challenge requires a realistic measure of the actual behavior and in-service performance. This characterization of behavior is essential to determine the actual load-carrying capacity or remaining capacity of a bridge, which is typi-

cally determined through a process called load rating. Load rating of a bridge defines the expected resistance or capacity based on its existing condition state and operating environment. As with bridge design, a challenge that exists for describing a bridge's capacity is the complex system interaction that exists amongst the superstructure components. For example, in a beam-slab bridge, the complexity is derived from the coupled interaction of two-way plate behavior within the bridge deck and the one-way beam behavior inherent to the girders. For both design and evaluation, a methodology for transverse distribution of loads, or live load distribution, is typically used to represent this phenomenon and provide a method to quantify relative load sharing behavior within the system.

In practice, this phenomenon is typically defined using prescriptive formulas that simplify the complex behavior into simple factors, but in recent decades, several refined methods for determining live load distribution have evolved. These methods provide alternative mechanisms to describe live load distribution behavior, which can often be more representative than the empirical methods included in most bridge design codes and specifications. The advantage of considering these methods is that they have the potential to describe the physical phenomenon and actual load distribution behavior, which in turn provides the bridge engineer with a mechanism to make more informed decisions regarding load restrictions, maintenance, and replacement of existing bridges.

1.2—Scope

This report is intended for the bridge engineering community, particularly engineers responsible for bridge load rating, to provide basic guidance on the methods and tools available for determining live load distribution behavior of in-service bridges. The objective is to present guidance on available methods for determining live load distribution, including approximate formulas, structural analysis, or load testing. The selection of a particular method of analysis is presented within the context of the intended level of refinement and bridge type, such as slab, beam-slab, and box girder. Included in this report are descriptions, a brief history, and background of the flexural load distribution phenomena and a summary of design and analysis methods used to describe the phenomena in practice. This report provides an overview of criteria for transverse load distribution, including their limitations and acceptability; a summary and description of the use of refined methods of analyses for transverse load distribution; and load test methods. A series of case studies are presented in the latter part of the report to serve as a comparative analysis of commonly used live load distribution methods and their performance in describing the behavior of in-service structures.

While this distribution phenomenon is relevant to a variety of force effects, this report focuses exclusively on flexure. The treatment of shear is a topic of future work by the committee and will be part of a guide on the in-service evaluation of concrete bridges, but is beyond the scope of this report. For a treatment of shear load distribution, the

reader is encouraged to review existing literature on the topic (Bakht et al. 1983; Ebeido and Kennedy 1995, 1996; Modjeski and Masters Inc. 2002; Barr and Amin 2006; Suksawang et al. 2013).

CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Notation

B	=	width of bridge, ft (m)
C	=	capacity
DC	=	dead load effect due to structural components and attachments
DF	=	lateral load distribution factor (also designated as g)
DW	=	dead load effect due to wearing surfaces and utilities
d_e	=	roadway part of overhang
F	=	amplification factor
IM	=	dynamic load allowance
k	=	number of girders
L	=	span of the bridge, ft (m)
LL	=	live load effect
$M_{g,ave}$	=	average moment per girder
m	=	multiple presence factor
n	=	number of wheel lines of applied loading
P	=	permanent loads other than dead loads
S	=	center-to-center spacing of longitudinal web lines or girders, ft (m); also spacing of floor beams, ft (m)
w_j	=	ratio section modulus of the i -th girder to section modulus of typical interior girder
ϵ_j	=	bottom flange strain at the i -th girder
γ_{DC}	=	LRFD load factor for structural components and attachments
γ_{DW}	=	LRFD load factor for wearing surfaces and utilities
γ_{LL}	=	evaluation live load factor
γ_P	=	LRFD load factor for permanent loads other than dead loads

2.2—Definitions

ACI provides a comprehensive list of definitions through an online resource, “ACI Concrete Terminology,” <https://www.concrete.org/store/productdetail.aspx?ItemID=CT13>. Definitions provided herein complement that resource.

beam slab—bridge system where the concrete slab or bridge deck is not supported by beams or girders and serves as the primary superstructure element or load-resisting component of the bridge.

composite beam-slab—member produced by interconnecting separate beam and slab; the connections have sufficient stiffness to ensure strain compatibility between components under service and sufficient strength to ensure the member strength is limited by material capacities in the individual components.

diaphragm—transverse member placed between the primary load-carrying elements of a superstructure system to distribute stresses and improve strength and rigidity.

dynamic load allowance (impact factor)—increase in the applied static live load force effects that account for the dynamic interaction between the bridge and moving vehicles.

effective width—reduced width of a concrete slab with an assumed uniform stress distribution, which produces the same effect on the behavior of a structural member as the actual slab with its non-uniform stress distribution.

finite difference method (differences)—method of analysis in which the governing differential equation is satisfied at discrete points on the structure.

finite element method (element)—method of analysis in which a structure is discretized into elements at nodes, the shape of the element as well as the displacement and/or stress fields are assumed, partial or complete compatibility is maintained among the element interfaces, and nodal displacements are determined by using energy variational principles or equilibrium methods.

floor beam—primary horizontal member spanning in the transverse direction relative to the general bridge alignment.

grillage analogy method—flexibility or stiffness method of analysis in which all or part of a superstructure is discretized into orthotropic planar grid components that represent the characteristics of the structure.

harmonic method—method of analysis in which the load model is subdivided into suitable parts, allowing each part to correspond to one term of a convergent infinite series by which structural deformations are described. See also **series method**.

lever rule—upper bound solution on the distribution of forces in a slab-on-girder bridge in which interior girders are assumed be hinged at the deck/girder intersection and a statical summation of moments about one point is used to calculate the reaction at a second point

live load distribution factor—fraction used to describe the portion of the total live load on the elements of a superstructure system, which represents a simplification of the complex bridge system behavior for design and analysis.

overhang—outward projection of a bridge slab that extends over the exterior beam.

refined method—method that considers the entire superstructure as one integral unit or system; for example, equivalent beam analysis, classical plate, grillage analogy analysis, three-dimensional frame analysis, and finite element analysis.

secondary member—member that does not carry calculated live loads or a member in which stress is not normally evaluated in the analyses; for example, bracing members and parapets.

series method—method of analysis in which the load model is subdivided into suitable parts, allowing each part to correspond to one term of a convergent infinite series by which structural deformations are described. See also **harmonic method**.

skew angle—angle measured between the axis of the substructure and a line perpendicular to the longitudinal axis of the superstructure.

slab-on-girder—bridge system in which a slab is placed on top of the girders, either composite or noncomposite with the girder.

spine—separate box on a multibox bridge.

transverse live load distribution—the process of transferring load from the deck to the girders through the transfer mechanism from the deck to the girders.

the bridge system into the primary load-carrying superstructure members or the load fraction that is resisted by an individual superstructure primary member.

CHAPTER 3—BASIS OF CODE CRITERIA FOR TRANSVERSE LIVE LOAD DISTRIBUTION

3.1—Introduction

This chapter discusses the limitations and applicability of the various bridge design codes and specifications described in the previous chapter, specifically the standard transverse live load distribution methods predominantly used in North America. To use the design codes and specifications, bridges must meet certain criteria. This includes limits on parameters such as span length, slab thickness, girder spacing, and longitudinal stiffness. The limits include both minimum and maximum values. The distribution of vehicular live load on bridges is a major factor in determining bridge capacity and serviceability, which are crucial parameters in determining safety and economy.

Where reference is made to the various design specifications, this document only highlights specific aspects of the specifications and the reader is encouraged to reference the specification directly. There has been a great deal of research on the accuracy of standard live load distribution methods used in bridge design codes and specifications (Khaleel and Itani 1990; Zokaie et al. 1991; Arockiasamy et al. 1997; Kim and Nowak 1997; Amer et al. 1999; Bridge Diagnostics Inc. 1999; Barney et al. 2000; Zokaie 2000; Barr et al. 2001; Conner and Santosuosso 2002; Wang and Puckett 2002; Song et al. 2003; Sotelino et al. 2004; Cai 2005; Puckett 2005; Harris 2010). These references provide additional technical guidance to the reader and can be used as a technical supplement to the summaries of the bridge design codes and specifications that are presented in this section. This chapter focuses primarily on the standard transverse live load distribution methods that are predominantly used in North America.

3.2—Transverse load distribution

Transverse live load distribution describes the load transfer mechanism from the vehicular live loads through the bridge system into the primary load-carrying superstructure members. Terms related and often interchanged with “transverse live load distribution” include “load distribution” and “distribution factors”. Transverse live load distribution is a function of bridge geometry, the connections between the various bridge elements, and the relative stiffness of components in both the transverse and longitudinal load directions.

Several methods exist for the determination of transverse live load distribution in bridges. The most commonly used methods are empirical formulas included in various bridge design codes and specifications (AASHTO 2002, 2011, 2014; CAN/CSA-S6; AREMA 2009). Numerical techniques that vary in sophistication are also used: beam analogy techniques, classical plate model, grillage analyses, and finite element analyses. In addition, load testing can be performed to directly measure load sharing behavior. The results from

load testing are recognized as the most accurate method for determining transverse live load distribution behavior, but these tests can be costly, challenging to execute, and are not applicable during the design phase. Similarly, refined methods of analysis have become well accepted within the bridge community and are proven to yield reliable results, but when compared to the use of empirical formulas, their implementation is typically more complex. Various forms of empirical formulas for describing live load distribution have been successfully used in design practice since the 1930s; these formulas are typically simple to implement and generally provide conservative and suitable estimates of the system behavior.

3.3—Empirical formulas for transverse live load distribution

Empirical equations are based predominantly on geometric factors such as girder spacing, span length, and deck thickness, and provide sufficient accuracy for most common bridge structures. The equations available within the North American codes can be classified as approximate methods and use an approach that simplified the complex system phenomena or live load distribution response into a distribution factor, which defines the relative force effect that must be designed for. These formulas for distribution factors are typically specific to a specific force effect (for example, moment or shear) and are often further categorized based on member location (that is, interior or exterior) and loading scenario (that is single or multiple truck loading). The following sections provide basic background information on the various bridge design codes used for concrete bridges in North America. This background information includes a summary of the general parameters used in the formulation, a limited history on their development, and a discussion on the limitations.

In general, bridges that have insufficient load capacity for legal truck loads based on approximate methods of analyses should be reanalyzed using more refined analytical tools to establish an acceptable capacity. In the case of skewed or irregularly shaped structures, actual distribution of live loads varies substantially from the distribution values determined using empirical equations and requires further modification to the basic form of the equations.

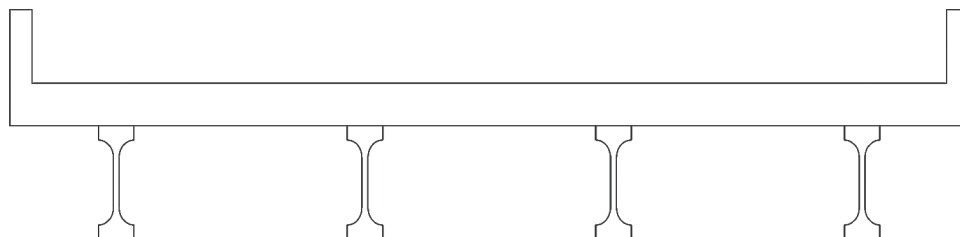
Identification of the most appropriate refined method of analysis for use depends on bridge configuration as well as bridge type. This is discussed in detail in subsequent chapters. If refined methods of analyses reveal that bridges have insufficient load capacity, then load testing may provide a more accurate determination of bridge stiffness, structural behavior, and live load sharing behavior.

3.4—AASHTO Standard Specification for Highway Bridges

Before the introduction of the AASHTO LRFD (1994), distribution factors used for design and evaluation were determined using the provisions of AASHTO Standard Specification for Highway Bridges (AASHTO 2002), which uses a simple ratio of girder spacing to a constant, often

Table 3.5—Summary of representative AASHTO (2014) distribution factors

Concrete deck, filled grid, partially filled grid, or unfilled grid deck composite with reinforced concrete slab on steel or concrete beams; concrete T-beams, and T- and double T-beams	One design lane loaded: $0.06 + \left(\frac{S}{14}\right)^{0.4} \left(\frac{S}{L}\right)^{0.3} \left(\frac{K_g}{12.0L^3}\right)^{0.1}$	Range of applicability $3.5 \leq S \leq 16.0$ $4.5 \leq t_s \leq 12.0$ $20 \leq L \leq 240$ $10,000 \leq K_g \leq 7,000,000$
	Two or more design lanes loaded: $0.075 + \left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_g}{12.0L^3}\right)^{0.1}$	

*Fig. 3.5.1a—Bridge cross section.*

referred to as the S-over method. These distribution factors from AASHTO (2002) were based on a wheel line concept or half of a truck load. The S-over method is based on bridge type, girder spacing range, and the number of traffic lanes. A representative example of the form of the equation type used in AASHTO (2002) would be $S/5.5$ for a concrete floor supported by four or more steel girders with a spacing between girders measuring less than 6 ft (1.8 m). Although the traditional factors were developed for simple, relatively short spans with small girder spacing, designers sometimes applied the distribution factors to continuous spans of significant proportions with relatively wide girder spacing. They were also used for bridges containing girders with variable spacing, curved girders, skewed supports, and virtually all kinds of plan geometry (AASHTO 2002), for which the equations may not be applicable.

3.4.1 Limitations—The AASHTO Standard Specification for Highway Bridges (AASHTO 2002) places few limitations on the use of the approximate formulas for lateral load distribution. Within the specification, the distribution factors are determined primarily based on the type of bridge, the girder spacing, number of traffic lanes, and girder spacing.

3.5—AASHTO LRFD Bridge Design Specifications

More comprehensive design equations (Zokaie et al. 1991; Zokaie 2000) for determining distribution factors were introduced in the first edition of the AASHTO LRFD Bridge Design Specifications. The load distribution factor equations in AASHTO (2014) are empirical and differ greatly from the S-over method prescribed in AASHTO (2002). The adopted equations were based on a National Cooperative Highway Research Program (1998) study aimed at developing simplified distribution factor design equations for shear and moment. The load and resistance factor design (LRFD) load distribution factors are grouped by bridge type and are generally functions of girder spacing S , span length L , slab thickness t_s , and beam stiffness K_g . The resulting equations include a parametric formulation that has good agreement

with rigorous analyses (Harris et al. 2010) and are based on modifications to the lever rule and Henry's method for load distribution (National Cooperative Highway Research Program 2007).

There are different equations for exterior and interior girders considering shear and moment for one-lane-loaded and two-or-more lanes-loaded cases. A representative set of equations for a typical girder bridge with a concrete deck is presented in Table 3.5 for moment on an interior beam. The complexity and the iterative nature of the new equations is more of an issue in design than in evaluation, as some of the parameters are not yet known.

3.5.1 Limitations—AASHTO (2014) is not applicable to every type and size of bridge. There are ranges of applicability in the specification to preserve the bridge's integrity and calibrated reliability index. There are several general conditions for using the load distribution provisions that are applicable to all bridges:

a) The bridge deck width cannot vary along its length. An example of the deck varying along the length would be if the bridge has two lanes in each direction with an exit lane at the end of the bridge. This bridge would be wider at the end and be designed using more refined methods such as finite element or finite strip grillage. The engineer can also use average beam spacing or maximum beam spacing for bridges with varying width.

b) The bridge cross section, unless otherwise specified, must have at least four beams or girders to support the bridge deck. Figure 3.5.1a shows an example bridge cross section. This cross section can have any type of girders: bulb-T, boxes, or I-beams. I-beams are shown as an example. There are a few exceptions to this where the minimum number is three, and these exceptions appear within the ranges of applicability for the relevant bridge types (refer to Table 4.6.2.2.2b-1 in AASHTO [2014]).

c) The spacing between parallel bridge beams or girders must remain constant throughout the length of the span and have approximately the same stiffness EI .

d) Unless otherwise specified, the roadway part of the overhang, d_e , does not exceed a specified value that depends on the structure type. The portion of the bridge roadway that extends beyond the exterior girders has both maximum and minimum values; that is, for a concrete slab on prestressed concrete girders, 5.5 ft (1.5 m) is the maximum allowed. The value can also be negative. Figure 3.5.1b shows a typical overhang section. The overhang distance does not include the width of the curb or barrier wall and is measured from the exterior web of the exterior beam to the interior faces of the curb or traffic barrier.

e) Curvature in plan is less than the limit specified in AASHTO (2014) and depends on the number of beams and spans and continuity. In this approach, a rational analysis of the entire superstructure is needed to proportion the computed moments, shears, and other force effects on the superstructure. All components of the superstructure should be considered as an integral structural unit. The live load distribution factors are computed with the exceptions noted in Tables 4.6.2.2.1-1, 4.6.2.2.2, and 4.6.2.2.3 of AASHTO (2014).

f) The cross section is consistent with those shown in Table 4.6.2.2.1-1 of AASHTO (2014): this table shows numerous typical cross sections and gives brief descriptions of the supporting components and deck types. Table 4.6.2.2.1-1 in AASHTO (2014) is the first table to check if the specification provisions for load distribution are applicable.

If a bridge configuration does not meet the requirements of AASHTO (2014), a refined analysis can be used to determine the live load distribution factors. AASHTO (2014), Sections 4.4 and 4.6.3, identify several different types of refined methods of analysis. Besides meeting all the general requirements stated previously, there are ranges of applicability that depend on the bridge type. For example, a concrete bulb-T-girder has different ranges of applicability than a multi-cell cast-in-place box girder. The range of applicability for the distribution factors also depends on whether the member is an interior or exterior girder and if the force effect is moment or shear. An example is the range for a concrete bulb-T for shear will be different than that corresponding range for moment. AASHTO (2014) includes ranges of applicability.

Most of the provisions are presented on the basis of a right or straight bridge configuration. When a bridge span or the primary longitudinal members are not perpendicular to the supports, the bridge is described as skewed. The skew angle of a bridge is defined as the angle measured between the axis of the substructure and a line perpendicular to the longitudinal axis of the superstructure (Precast/Prestressed Concrete Institute 2014). The skew of a bridge is known to affect the distribution of both moments and shears, and AASHTO (2014) includes correction factors to account for this effect. For skewed bridges, a reduction of the moment is an option, but only if the difference in skew angles between all the supports within the bridge does not exceed 10 degrees. AASHTO (2014) Table 4.6.2.2.2e-1 shows the range of applicability for moment reduction on skewed bridges. If

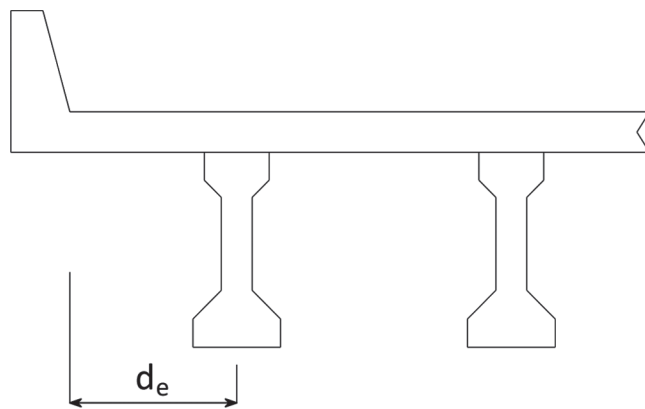


Fig. 3.5.1b—Deck overhang.

the bridge is not covered in any of the cases in the table, no reduction factors are available.

Per AASHTO (2014), if the deck is directly supported by transverse floor beams with a spacing S (in ft [m]) that does not exceed 6 ft (1.8 m), then the fraction of the load distributed to the floor beam is $S/6$ of the 32 kip (144 kN) design axle load. If the spacing exceeds 6 ft (1.8 m), the total design loads are considered and the lever rule is applied.

3.6—Canadian Highway Bridge Design Code

CAN/CSA-S6 specifies analysis requirements for many bridge superstructure types. For short- and medium-span concrete bridges, these superstructure types include slab and voided slab bridges, slab-on-girder bridges, arch bridges, rigid frames with integral abutments bridges, and box girder bridges.

For each bridge type, CAN/CSA-S6 specifies the appropriate structural response parameters to consider in design. These response parameters are longitudinal moment, transverse moment, longitudinal torsion, longitudinal vertical shear, transverse vertical shear, and in-plane forces. The clauses within CAN/CSA-S6 provide further guidance on additional factors such as cross bracing or variation of cross section, which can be incorporated in the modeling and analysis. CAN/CSA-S6 also permits refined methods and simplified methods to determine the structural response to both dead and live loads.

The refined methods of analysis include grillage analogy, orthotropic plate theory, finite element, finite strip, folded plate, and semi-continuum. CAN/CSA-S6 provides further guidance with respect to refined methods appropriate to particular superstructure types and formula for determining key geometric and material parameters for the various methods of analysis.

If a bridge superstructure type satisfies certain criteria, then CAN/CSA-S6 permits the use of more simplified methods of analysis. The simplified methods are used to determine the longitudinal moment and longitudinal vertical shear for the limit states of ultimate, serviceability, and fatigue. The method has evolved from methods based on the orthotropic plate theory that first appeared in the 1979 Ontario Highway Bridge Design Code (Ontario Ministry of Transportation and

Communications 1979) and described in the corresponding commentary.

The conditions for the use of simplified methods of analysis are very similar to those specified in AASHTO (2014). They include criteria for the regularity of the superstructure geometry, support conditions, number of girders, bridge skew, and length of deck slab overhangs.

The simplified method in CAN/CSA-S6 is based on determining an amplification factor F . This factor accounts for the transverse variation in maximum longitudinal moment or shear compared to the average longitudinal moment or vertical shear intensity. For example, $M_{g,ave}$ is the average moment per girder due to live load determined by distributing the total moment on the bridge cross section equally among all girders with consideration of the number of design lanes and multiple presence probability.

CAN/CSA-S6 presents various tables and equations to determine amplification factors for different superstructure types and limit states. This process considers the number of girders, design lane width, bridge width, bridge span, fitting parameters for simplified analysis, and a coefficient for transverse distribution of load effects in the bridge. This process determines girder design moments for both interior and exterior girders. The longitudinal design shears are determined in a similar manner. A key difference between the CAN/CSA-S6 and AASHTO (2002) and AASHTO (2014) is that CAN/CSA-S6 yields a design moment for member proportioning, whereas the AASHTO provisions must still be applied to the applied live load moment.

3.6.1 Limitations—For dead load analysis, the beam analogy method specified in CAN/CSA-S6-06, 5.6.1.2, may be used for bridges satisfying the following conditions:

- a) The width of the deck is constant.
- b) Support conditions are closely equivalent to line support at the ends, and in the case of multi-span bridges, at intermediate supports.
- c) For slab and voided slab bridges, the skew parameter, $\varepsilon = B(\tan\psi)/L$, does not exceed 1/6, and for slab-on-girder bridges built with shored construction, the skew parameter, $\varepsilon = S(\tan\psi)/L$, does not exceed 1/18, where S is the center-to-center spacing of longitudinal web lines or girders in ft (m), B is the width of the bridge in ft (m), L is the span of the bridge in ft (m), and ψ is the skew angle in degrees. For slab-on-girder bridges built with unshored construction, there is no limitation on the value of the skew parameter applied to the dead load.
- d) For bridges that are curved in plan and built with shored construction, the radius of curvature, span, and width should satisfy the requirements of CAN/CSA-S6-06, A5.1.3.2.
- e) Slab and voided slab bridges are either of substantially uniform depth across a transverse section or tapered in the vicinity of a free edge, provided that the length of the taper in the transverse direction does not exceed 8 ft (2 m).
- f) For a bridge with longitudinal girders and an overhanging deck slab, the overhang is not more than 6 ft (1.8 m) and does not exceed 60 percent of the mean spacing between the longitudinal girders or, for box girder bridges, 60 percent of the spacing of the two outermost adjacent webs.

When these conditions are not fully met, engineering judgment should be used to determine whether the bridge satisfies these conditions to an extent sufficient for the beam analogy method to apply.

For live load analysis, the simplified methods of CAN/CSA-S6-06, 5.7.1.2 to 5.7.1.5, may be used for bridges satisfying the following conditions:

- a) The width of the deck is constant.
- b) Support conditions are closely equivalent to line support at the ends of the bridge and, in the case of multi-span bridges, at intermediate supports.
- c) For skewed slab bridges and slab-on-girder bridges, the skew parameter requirements of CAN/CSA-S6-06, A5.1.3.1, are met.
- d) For bridges that are curved in plan, the radius of curvature, span, and width satisfy the applicable requirements of CAN/CSA-S6-06, A5.1.3.2.
- e) Slab and voided slab bridges are either of substantially uniform depth across a transverse section or tapered in the vicinity of a free edge, provided that the length of the taper in the transverse direction does not exceed 8 ft (2.5 m).
- f) For slab-on-girder bridges, there are at least three longitudinal girders of equal flexural rigidity and equal spacing, or with variations from the mean in rigidity and spacing of not more than 10 percent in each case.
- g) For multi-spine bridges, there are at least two longitudinal girders of nearly equal flexural rigidity and equal spacing.
- h) For bridges with longitudinal girders and an overhanging deck slab, the overhang does not exceed 6 ft (1.8 m) and does not exceed 60 percent of the mean spacing between the longitudinal girders. For box girder bridges, the overhang does not exceed 6 ft (1.8 m) and does not exceed 60 percent of the spacing of the two outermost adjacent webs.
- i) For continuous span bridges, the requirements of CAN/CSA-S6-06, A5.1.2, should be met.
- j) For multi-spine bridges, each spine has only two webs, and, for steel and steel-composite multi-spine bridges, the requirements of CAN/CSA-S6-06, 10.12.5.1, should be met.
- k) For multi-spine bridges, the number of lanes is at least equal to the number of spines and B/L is at least equal to 1.0.

When these conditions are not fully met, engineering judgment should be used to determine whether the bridge satisfies these conditions to an extent sufficient for the appropriate simplified method to apply. For the purpose of using simplified methods of analysis for live load, superstructure types are categorized as specified in Table 5.2 of CAN/CSA-S6-06.

3.7—American Railway Engineering and Maintenance-of-Way Association

Railroad bridges are known for their longevity. This is due in part to the conservatism in design that is prevalent in AREMA (2009) and decreases in railroad loads as locomotives transitioned from steam-driven to diesel-powered. Given that knowledge, a detailed comparison of highway and railroad bridges reveals more differences than similarities.

ties in their load and load distribution requirements. This is because:

a) Live load-to-dead load ratio is greater for railroad bridges.

b) Frequency of maximum design loading is greater in railroad bridges, especially bridges that carry heavily loaded unit trains, defined as trains transporting a single commodity from one source to one destination.

c) Railroad track is fixed in position and lateral distribution is based on the crossties and their ability to support the wheel load.

d) Axle loads are uniformly distributed longitudinally over a length of 3 ft (0.9 m) plus the depth of ballast under the tie plus twice the effective depth of the slab, but limited to the axle spacing.

For the longitudinal members of ballasted deck structures (common for concrete bridges), the live load distribution is based on tests performed by the Association of American Railroads (Sanders and Muse 1969) and is assumed uniform over a width equal to the length of track tie plus the depth of ballast and fill below the bottom of the tie, unless limited by the extent of the structure. This requirement assumes that load is distributed beyond the width of the ties at vertical-to-horizontal slope of 2:1 through the ballast. Unlike highway bridges, railroad bridges maintain a more consistent distribution of loads due to the fixed track structure that controls the transverse position of the live loads.

3.7.1 Limitations—AREMA (2009) does not allow for transverse distribution of track loadings beyond the width of the ties plus the depth of the ballast. The transverse distribution of live load through ballast is limited by the width of the structure.

3.8—Summary

To describe the complex interaction and load sharing behavior of members within a bridge system, the concept of live load distribution is typically used. This concept describes the load transfer mechanism from the vehicular live loads through the bridge system into the primary load-carrying superstructure members.

Design provisions and specifications used in North America include different empirical and semi-empirical relationships to describe this concept, but all of these methods represent simplifications of the actual complex phenomena. Prior to the implementation of the AASHTO LRFD specification in 1994 (AASHTO 1994), the AASHTO Standard Specifications for Highway Bridges (AASHTO 2002) simple ratio formula (S-over) had been used to determine live load distribution factors since the 1930s. AASHTO (2002) was used for most common cases for calculating the bending moments and shears in bridge design, with few limitations imposed. AASHTO (2014) and CAN/CSA-S6 include more detailed formulas for determining transverse live load distribution factors that provide distinctions between bridge type, location, and loading scenario with more specific limits on the range of applicability of the formulas. For railroad bridges, the distribution of live loads is much less complex due to

the nature of the fixed track location, and AREMA (2009) maintains prescriptive guidelines on live load distribution.

Within the range of applicability, these code-defined distribution factors are intended to yield conservative results for design, but when extended to evaluation of existing bridges, these methods have the potential to yield either conservative or unconservative results. It should be noted that code-defined distribution factors were developed based on intact bridges with little or no physical deterioration; therefore, the physical condition of a bridge must be accounted for in the bridge evaluation. In these cases, it may be appropriate to use a more refined method (refer to Chapter 4) of analysis to describe the load distribution behavior. A general discussion on the evaluation of bridge condition and its influence on behavior is included in Chapter 6; however, the reader is encouraged to consult additional literature for more comprehensive coverage (Federal Highway Administration 2006; ACI 228.2R).

CHAPTER 4—SUMMARY AND USE OF REFINED METHODS OF ANALYSIS

4.1—Introduction

The selection of an appropriate structural analysis method for the purpose of determining transverse load distribution factors is critical to understanding the behavior of a bridge system. Engineers should evaluate the trade-off between accuracy of the results and the level of complexity associated with the analysis. A variety of structural analysis methods and supporting software are available, and they range from easy-to-use approximate methods to highly sophisticated and refined methods that include numerical and analytical methods.

AASHTO (2014) provides some guidelines to select an analysis method for the design of new bridges and presents some acceptable analysis methods. The acceptable methods listed in 4.4 of AASHTO (2014) include any method of analysis that satisfies the requirements of equilibrium and compatibility, and uses the appropriate stress-strain relationships for the materials used. The methods can be grouped into three general categories: 1) classical methods; 2) numerical methods; and 3) analytical methods.

4.1.1 Classical methods—Classical force and displacement methods can incorporate equivalent strip for deck slab, effective flange width for box section, and moment amplification factors to account for the beam-column effect. Classical methods are suitable for routine analysis of typical bridge types. Most importantly, the classical methods are easy to use and require less time and effort than sophisticated methods during the evaluation and design phase.

4.1.2 Numerical methods—Numerical methods include finite difference, finite element method, finite strip method, and grillage analogy. Numerical methods are based on dividing the integrated structure into discrete members. The displacements, reactions, forces, stresses, or all, in the structure are obtained through numerical analysis instead of closed-form analysis. The main advantage of numerical methods is the computational efficiency. Virtually all

commercial structural analysis programs use numerical methods and the evolution of computational resources have greatly enhanced the capability of numerical methods to solve increasing complex systems in terms of the number of redundancies, scale of structures, and number of load cases, which were historically the restrictions when applying these methods. The accuracy of numerical method results depends highly on the accuracy of the simulation technique, modeling assumptions and simplifications. Inadequacies such as inappropriate structural modeling, incorrect boundary conditions, improper material characteristics, or inaccurate member displacement/force continuity can lead to erroneous results. The use of numerical methods and software generally requires experienced analysts.

4.1.3 Analytical methods—Analytical methods include the series or other harmonic methods, yield line theory, and the folded plate method. These methods are generally derived from classical theories of mechanics with some assumptions and simplifications. The solutions are generally in closed forms, and are therefore easy to use. Generally speaking, analytical methods are highly accurate, but have limited applicability in characterizing load distribution behavior.

4.1.4 Summary—In general, each analysis method has its own advantages and limitations, and should be selected in consideration of the different objectives and desired results. As it relates to lateral load distribution behavior, only a limited number of these refined methods are relevant. These methods include classical force and displacement methods, finite difference method, finite element method, grillage analysis, and classical or orthotropic plate theory. An argument can be made that other methods are applicable, but those applications are far less common.

4.2—General approach

A realistic understanding of the load distribution mechanism—particularly vehicular live loads on a bridge superstructure—using refined methods of analysis, is the focus of this section. Bridges with insufficient load capacity based on an empirical formula for live load distribution, or conditions outside the scope of such methods, require more refined analytical tools to accurately evaluate actual load distribution behavior. Examples of bridge conditions that require refined methods of analysis include the following:

- a) A low load rating
- b) Permit loads with nonstandard wheel-gauge widths and axle configurations
- c) Girder spacing and span lengths outside the limits of approximate distribution formulas
- d) Bridges with sharp skew angles (more than 45 to 60 degrees) or varying skew angles
- e) Nontraditional framing of girders and floor beams
- f) Curved bridges
- g) Bridges with observed or measured damage, deterioration, or both
- h) Bridges with innovative or nontraditional designs and new construction technologies

Selection of a refined method of analysis for a particular bridge in the context of the bridge type and the intended level

of refinement is critical in the evaluation stage. A typical approach for assessment would use the simplest analytical tool as the initial method along with the most conservative assumptions. This can be followed by higher order methods and criteria, if necessary. As an example, if the model is not able to describe the bridge behavior properly or if the initial analysis does not meet the expectations of the bridge owner, use of a more refined method of analysis may be appropriate. Concrete bridges and bridge members are generally categorized based on superstructure characteristics—beam slab (including steel beams with concrete deck), arch and rigid frame, slab, and box girder bridges—and can be analyzed using a variety of methods. Among the various methods previously described, the equivalent beam, grillage analogy, three-dimensional frame, and finite element analysis methods are discussed in this chapter. The grillage analogy and three-dimensional frame analysis methods are classified as more traditional methods and are not typically used in current practice, but provide a rational analysis tool that can be implemented using less-complex software packages than the more commonly used finite element analysis method.

Influence lines or surfaces can be used within the context of each of these methods of analysis, especially for the more refined ones, to determine worst-case longitudinal and transverse loading scenarios on a bridge. This approach is necessary when the available analysis software is limited by the maximum number of live load cases allowed or when a definitive bounding solution by direct application of axle loads, lane loads, or both, is not possible.

For most analyses, linear elastic material behavior is typically assumed with any of the previously listed methods of analysis; however, nonlinear analyses might be appropriate when the potential for inelastic deformations and significant deterioration exist. In all of the modeling approaches, the user should be able to extract critical force effect and deformation information used to describe the load distribution behavior of the system. For transverse live load distribution, this can be in the form of critical moments in the member, but the models can also be used to determine deformations such as deflections or strains. A measure of this internal load-sharing behavior is critical to the design and load rating processes, and is used to determine the overall bridge capacity or load rating.

4.3—Equivalent beam line method

The equivalent beam method is based on classical force and displacement methods of analysis where the bridge system is simplified down to a representative single beam. This approach is most commonly used for beam-slab bridges, which consist of multiple longitudinal beams or girders made of concrete or steel supporting a concrete deck slab. The approach can be extended to a two-dimensional frame analysis if the substructure elements are included in the model. Typical commercially available structural analysis software packages can be used for the analyses.

4.3.1 Applicability—The equivalent beam line method is appropriate for slab and beam-slab bridges, multi-cell box and adjacent box beam bridges, multi-cell box, and segmental