

Analysis and Design of Reinforced Concrete Bridge Structures

Reported by Joint ACI-ASCE Committee 343

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These recommendations, reported by Joint ACI-ASCE Committee 343 on Concrete Bridge Design, provide currently acceptable guidelines for the analysis and design of reinforced, prestressed, and partially prestressed concrete bridges based on the state of the art at the time of writing the report. The provisions recommended herein apply to pedestrian bridges, highway bridges, railroad bridges, airport taxiway bridges, and other special bridge structures. Recommendations for Transit Guideways are given in ACI 358R.

The subjects covered in these recommendations are: common terms; general considerations; materials; construction; loads and load combinations; preliminary design; ultimate load analysis and strength design; service load analysis and design; prestressed concrete; superstructure systems and elements; substructure systems and elements; precast concrete; and details of reinforcement.

The quality and testing of materials used in construction are covered by reference to the appropriate AASHTO and ASTM standard specifications. Welding of reinforcement is covered by reference to the appropriate AWS standard.

Keywords: admixtures; aggregates; anchorage (structural); beam-column frame; beams (supports); **bridges (structures)**; cements; cold weather construction; columns (supports); combined stress; composite construction (concrete and steel); composite construction (concrete to concrete); compressive strength; **concrete construction**; concretes; concrete slabs;

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construction joints; **construction materials**; continuity (structural); cover; curing; deep beams; deflection; earthquake-resistant structures; flexural strength; footings; **formwork (construction)**; frames; hot weather construction; **inspection**; lightweight concretes; **loads (forces)**; mixing; mixture proportioning; modulus of elasticity; moments; **placing**; **precast concrete**; prestressed concrete; prestressing steels; **quality control**; **reinforced concrete**; reinforcing steels; **serviceability**; shear strength; spans; **specifications**; splicing; strength; **structural analysis**; **structural design**; T-beams; torsion; ultimate strength method; water; welded-wire fabric.

Note: In the text, measurements in metric (SI) units in parentheses follow measurements in inch-pound units. Where applicable for equations, equations for metric (SI) units in parentheses follow equations in inch-pound units.

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CHAPTER 1—DEFINITIONS, NOTATION, AND ORGANIZATIONS

1.1—Introduction

This chapter provides currently accepted definitions, notation, and abbreviations particular to concrete bridge design practice which have been used in the preparation of this document.

Concrete bridge types commonly in use are described separately in Chapter 2, Requirements for Bridges, in Chapter 6, Preliminary Design, and in Chapter 11, Superstructure Systems and Elements.

1.2—Definitions

For cement and concrete terminology already defined, reference is made to ACI 116R. Terms not defined in ACI 116R or defined differently from ACI 116R are defined for specific use in this document as follows:

aggregate, normal weight—aggregate with combined dry, loose weight, varying from 110 lb. to 130 lb/ft³ (approximately 1760 to 2080 kg/m³).

compressive strength of concrete (f'_c)—specified compressive strength of concrete in pounds per square inch (psi) or (MPa).

Wherever this quantity is under a radical sign, the square root of the numerical value only is intended and the resultant is in pounds per square inch (psi) or (MPa).

concrete, heavyweight—a concrete having heavyweight aggregates and weighing after hardening over 160 lb/ft³ (approximately 2560 kg/m³).

concrete, shrinkage-compensating—an expansive cement concrete in which expansion, if restrained, induces

compressive strains that are intended to approximately offset tensile strains in the concrete induced by drying shrinkage.

concrete, structural lightweight—concrete containing lightweight aggregate having unit weight ranging from 90 to 115 lb/ft³ (1440 to 1850 kg/m³). In this document, a lightweight concrete without natural sand is termed “all-lightweight concrete,” and lightweight concrete in which all fine aggregate consists of normal weight sand is termed “sand-lightweight concrete.”

design load—applicable loads and forces or their related internal moments and forces used to proportion members. For service load analysis and design, design load refers to loads without load factors. For ultimate load analysis and strength design, design load refers to loads multiplied by appropriate load factors.

effective prestress—the stress remaining in concrete due to prestressing after all losses have occurred, excluding the effect of superimposed loads and weight of member.

load, dead—the dead weight supported by a member (without load factors).

load, live—the live load specified by the applicable document governing design (without load factors).

load, service—live and dead loads (without load factors).

plain reinforcement—reinforcement without surface deformations, or one having deformations that do not conform to the applicable requirements for deformed reinforcement.

pretensioning—a method of prestressing in which the tendons are tensioned before the concrete is placed.

surface water—water carried by an aggregate, except that held by absorption within the aggregate particles themselves.

1.3—Notation

Preparation of notation is based on ACI 104R. Where the same notation is used for more than one term, the uncommonly used terms are referred to the Chapter in which they are used. The following notations are listed for specific use in this report:

a	= depth of equivalent rectangular stress block
a	= constant used in estimating unit structure dead load (Chapter 5)
a	= compression flange thickness (Chapter 7)
a_b	= depth of equivalent rectangular stress block for balanced conditions
a_i	= fraction of trucks with a specific gross weight
a_v	= ratio of stiffness of shearhead arm to surrounding composite slab section
A	= effective tension area of concrete surrounding the main tension reinforcing bars and having the same centroid as that reinforcement, divided by the number of bars, or wires. When the main reinforcement consists of several bar or wire sizes, the number of bars or wires should be computed as the total steel area divided by the area of the largest bar or wire used
A	= axial load deformations and rib shortening used in connection with t -loads (Chapter 5)
A_b	= area of an individual bar
A_c	= area of core of spirally reinforced compression member measured to the outside diameter of the spiral
A_e	= area of longitudinal bars required to resist torsion
A_e	= effective tension area of concrete along side face of member surrounding the crack control reinforcement (Chapter 8)
A_f	= area of reinforcement required to resist moment developed by shear on a bracket or corbel
A_g	= gross area of section
A_h	= area of shear reinforcement parallel to flexural tension reinforcement
A_l	= total area of longitudinal reinforcement to resist torsion
A_n	= area of reinforcement in bracket or corbel resisting tensile force N_{uc}
A_{ps}	= area of prestressed reinforcement in tension zone
A_s	= area of tension reinforcement
A_s'	= area of compression reinforcement
A_{sa}	= area of bonded reinforcement in tension zone
A_{se}	= area of stirrups transverse to potential bursting crack and within a distance s_e
A_{sf}	= area of reinforcement to develop compressive strength of overhanging flanges of I- and T-sections
A_{sh}	= total area of hoop and supplementary cross ties in rectangular columns
A_{st}	= total area of longitudinal reinforcement (in compression members)

A_t	= area of one leg of a closed stirrup resisting torsion within a distance s
A_v	= area of shear reinforcement within a distance s , or area of shear reinforcement perpendicular to flexural tension reinforcement within a distance s , for deep flexural members
A_{vf}	= area of shear-friction reinforcement
A_{vh}	= area of shear reinforcement parallel to the flexural tension reinforcement within a distance s_2
A_w	= area of an individual wire
A_1	= loaded area, bearing directly on concrete
A_2	= maximum area of the portion of the supporting surface that is geometrically similar to, and concentric with, the loaded area
b	= width of compressive face of member
b	= constant used in estimating unit structure dead load (Chapter 5)
b	= width or diameter of pier at level of ice action (Chapter 5)
b	= width of web (Chapter 6)
b	= width of section under consideration (Chapter 7)
b_e	= width of concrete section in plane of potential bursting crack
b_o	= periphery of critical section for slabs and footings
b_v	= width of the cross section being investigated for horizontal shear
b_w	= web width, or diameter of circular section
B	= buoyancy
c	= distance from extreme compressive fiber to neutral axis
C	= construction, handling, and erection loads (Chapter 5)
C	= stiffness parameter used in connection with lateral distribution of wheel loads to multibeam precast concrete bridges (Chapter 10)
C	= ultimate creep coefficient (Chapter 5)
C_a	= indentation coefficient used in connection with ice forces
C_e	= exposure coefficient used in connection with wind forces
C_i	= coefficient for pier inclination from vertical
C_m	= factor used in determining effect of bracing on columns (Chapter 7)
C_t	= factor relating shear and torsional stress properties equal to b_w times d divided by the summation of x^2 times y
C_t	= creep deformation with respect to time (Chapter 5)
C_u	= ultimate creep deformation (Chapter 5)
C_u	= ultimate creep coefficient
C_w	= shape factor relating to configuration of structure and magnitude of wind force on structure
CF	= centrifugal force
d	= distance from extreme compressive fiber to centroid of tension reinforcement
d	= depth of section under consideration (Chapter 7)
d	= depth of girder (Chapter 5)
d'	= distance from extreme compressive fiber to

	centroid of compression reinforcement	f_{cir}	= concrete stress immediately after transfer at center of gravity of prestressing steel
d_b	= nominal diameter of bar, wire, or prestressing strand	f_{cp}	= concrete bearing stress under anchor plate of post-tensioning tendon
d_c	= thickness of concrete cover measured from the extreme tensile fiber to the center of the bar located closest thereto	f_{ct}	= average splitting tensile strength of lightweight aggregate concrete
d_p	= effective depth of prestressing steel (Chapter 7)	f_f	= stress range
d_s	= effective depth for balanced strain conditions (Chapter 7)	f_l	= stress produced by i th loading (Chapter 5)
d_u	= effective depth used in connection with prestressed concrete members (Chapter 7)	f_{lc}	= loss in prestressing steel stress due to creep
D	= dead load	f_{le}	= loss in prestressing steel stress due to elastic shortening
D	= diameter of lead plug in square or circular isolation bearing (Chapter 11)	f_{lf}	= loss in prestressing steel stress due to friction
D_f	= depth of footing	f_{lp}	= total loss in prestressing steel stress
DF	= distribution factor used in connection with live loads	f_{lr}	= loss in prestressing steel stress due to relaxation
DR	= derailment force	f_{ls}	= loss in prestressing steel stress due to shrinkage
DS	= displacement of supports	f_{min}	= algebraic minimum stress level where tension is positive and compression is negative
e	= base of Napierian logarithms	f_{pc}	= compressive stress in the concrete, after all prestress losses have occurred, at the centroid of the cross section resisting the applied loads or at the junction of the web and flange when the centroid lies in the flange. (In a composite member, f_{pc} will be the resultant compressive stress at the centroid of the composite section, or at the junction of the web and flange when the centroid lies within the flange, due to both prestress and to bending moments resisted by the precast member acting alone)
e	= span for simply supported bridge or distance between points of inflection under uniform load (Chapter 10)	f_{pe}	= compressive stress in concrete due to prestress only, after all losses, at the extreme fiber of a section at which tensile stresses are caused by applied loads
e	= eccentricity of design load parallel to axis measured from the centroid of the section (Chapter 7)	f_{po}	= steel stress at jacking end of post-tensioning tendon
e_b	= M_b/P_b = eccentricity of the balanced condition-load moment relationship	f_{ps}	= stress in prestressing steel at design loads
e_n	= clear span length of slab (Chapter 10)	f_{pu}	= ultimate strength of prestressing steel
e_1	= length of short span of slab	f_{py}	= specified yield strength of prestressing tendons
e_2	= length of long span of slab	f_r	= modulus of rupture of concrete
E	= effective width of concrete slab resisting wheel or other concentrated load (Chapter 10)	f_s	= tensile stress in reinforcement at service loads
E	= earth pressure used in connection with loads (Chapter 5)	f'_s	= stress in compressive reinforcement
E_c	= modulus of elasticity of concrete	f'_{sb}	= stress in compressive reinforcement at balanced conditions
E_{ci}	= modulus of elasticity of concrete at transfer of stress	f_{se}	= effective stress in prestressing steel, after losses
E_{ps}	= modulus of elasticity of prestressing strand	f_t	= extreme fiber tensile stress in concrete at service loads
E_s	= modulus of elasticity of steel	f_y	= specified yield stress, or design yield stress of nonprestressed reinforcement
EI	= flexural stiffness of compression members	f_{ya}	= design yield stress of steel of bearing plate
EQ	= earthquake force	f_{yh}	= design yield stress of steel for hoops and supplementary cross ties in columns
f	= natural frequency of vibration of structure (Chapter 5)	F	= frictional force
f_a	= axial stress	F	= horizontal ice force on pier (Chapter 5)
f_a	= basic allowable stress (Chapter 5)	F_a	= allowable compressive stress
f_b	= bending stress	F_b	= allowable bending stress
f_b	= average bearing stress in concrete on loaded area (Chapter 8)	g	= acceleration due to gravity, 32.2 ft/sec ² (9.81 m/sec ²)
f_c	= extreme fiber compressive stress in concrete at service loads	G_A	= ratio of stiffness of column to stiffness of members at A end resisting column bending
f'_c	= specified compressive strength of concrete		
f_{cds}	= change in concrete stress at center of gravity of prestressing steel due to all dead loads except the dead load acting at the time the prestressing force is applied		
f'_{ci}	= compressive strength of concrete at time of initial prestress		

G_A	= degree of fixity in the foundation (Chapter 11)	l_a	= additional embedment length at support or at point of inflection
G_B	= ratio of stiffness of column to stiffness of members at B end resisting column bending	l_a	= distance from face of support to load for brackets and corbels (Chapter 7)
G_{avg}	= average ratio of stiffness of column to stiffness of members resisting column bending	l_{bd}	= basic development length for deformed bar in compression
G_{min}	= minimum ratio of stiffness of column to stiffness of members resisting column bending	l_d	= development length
h	= overall thickness of member	l_{dh}	= development length for deformed bars in tension terminating in a standard hook
h	= slab thickness (Chapter 6)	l_{hb}	= basic development length of hooked bar
h	= height of rolled on transverse deformation of deformed bar (Chapter 8)	l_n	= clear span measured face-to-face of supports
h	= height of fill (Chapter 5)	l_n	= length of tendon (Chapter 3)
h	= thickness of ice in contact with pier (Chapter 5)	l_u	= unsupported length of compression member
h	= asphalt wearing surface thickness (Chapter 5)	L	= live load
h_a	= thickness of bearing plate	L	= span length used in estimating unit structure dead load (Chapter 5)
h_c	= core dimension of column in direction under consideration	L	= bridge length contributing to seismic forces (Chapter 5)
h_f	= compression flange thickness of I- and T-sections	L	= length of compression member used in computing pier stiffness (Chapter 11)
h_o	= thickness of standard slab used in computing shrinkage	LF	= longitudinal force from live load
h_2	= thickness of bottom slab of box girder (Chapter 6)	M	= number of individual loads in the load combination considered
H	= average height of columns supporting bridge deck	M	= live load moment per unit width of concrete deck slab (Chapter 10)
H	= curvature coefficient (Chapter 9)	M_a	= maximum moment in member at stage for which deflection is being computed
I	= impact due to live load (Chapter 5)	M_b	= nominal moment strength of a section at simultaneous assumed ultimate strain of concrete and yielding of tension reinforcement (balanced conditions)
I	= impact coefficient	M_c	= factored moment to be used for design of compression member
I	= moment of inertia (Chapter 7)	M_{cr}	= moment causing flexural cracking at sections due to externally applied loads
ICE	= ice pressure	M_m	= modified moment (Chapter 7)
I_{cr}	= moment of inertia of cracked section with reinforcement transformed to concrete	M_{max}	= maximum factored moment due to externally applied loads, dead load excluded
I_e	= effective moment of inertia for computation of deflection (Chapter 8)	M_n	= nominal moment strength of section
I_g	= moment of inertia of gross concrete section about the centroidal axis, neglecting the reinforcement	M_{nx}	= nominal moment strength of section about x -axis
I_s	= moment of inertia of reinforcement about the centroidal axis of the member cross section	M_{ny}	= nominal moment strength of section about y -axis
k	= effective length factor for compression member (Chapters 7 and 11)	M_u	= factored moment at section, $M_u = \phi M_n$
k	= dimensionless coefficient for lateral distribution of live load for T- and I-girder bridge (Chapter 10)	M_{ux}	= factored moment at section about x -axis, $M_{ux} = \phi M_{nx}$
k	= coefficient for different supports in determining earthquake force (Chapter 5)	M_{uy}	= factored moment at section about y -axis, $M_{uy} = \phi M_{ny}$
k_e	= dimensionless coefficient for lateral distribution of live load for spread box-beam bridges (Chapter 10)	M_x	= applied design moment component about x -axis
K	= wobble friction coefficient of prestressing steel (Chapter 9)	M_y	= applied design moment component about y -axis
K	= constant used in connection with stream flow (Chapters 5 and 11)	M_1	= value of smaller factored end moment on compression member calculated from a conventional or elastic analysis, positive if member is bent in single curvature, negative if bent in double curvature
K	= value used for beam type and deck material (Chapter 10)	M_2	= value of larger factored end moment on compression member calculated by elastic analysis, always positive
K	= pier stiffness (Chapter 11)	n	= modular ratio E_s/E_c
l	= length		

n	= number of individual loads in the load combination considered (Chapter 5)	r	= radius of gyration of the cross section of compression member
n_b	= number of girders (Chapter 10)	r	= base radius of rolled on transverse deformation of deformed bar (Chapter 8)
n_e	= number of design traffic lanes (Chapter 10)	R	= average annual ambient relative humidity, percent
N	= nosing and lurching force	R_n	= characteristic strength (moment, shear, axial load)
N	= minimum support length (Chapter 5)	RH	= mean annual relative humidity, percent (Chapter 5)
NB	= number of beams	s	= shear or torsion reinforcement spacing in direction parallel to longitudinal reinforcement
NL	= number of design traffic lanes	s	= beam spacing (Chapter 6)
N_u	= design axial load normal to the cross section occurring simultaneously with V_u , to be taken as positive for compression, negative for tension, and to include the effects of tension due to shrinkage and creep	s_e	= spacing of bursting stirrups
N_{uc}	= factored tensile force applied at top of bracket or corbel acting simultaneously with V_u , taken as positive for tension	s_2	= shear or torsion reinforcement spacing in direction perpendicular to the longitudinal reinforcement or spacing of horizontal reinforcement in wall
o	= overhang of bridge deck beyond supporting member (Chapter 6)	s_w	= spacing of wires
o	= effective ice strength (Chapter 5)	S	= span length
OL	= overload	S	= average beam spacing for distribution of live loads (Chapter 10)
p	= allowable bearing	S	= shrinkage and other volume changes used in connection with loads or forces to be considered in analysis and design (Chapter 5)
p	= minimum ratio of bonded reinforcement in tension zone to gross area of concrete section (Chapter 9)	S_h	= vertical spacing of hoops (stirrups) with a maximum of 4 in. (Chapter 11)
p	= unit weight of air (Chapter 5)	S_h	= spacing of hoops and supplementary cross ties
p	= proportion of load carried by short span of two-way slab (Chapter 10)	SF	= stream flow pressure = KV^2
P	= load on one rear wheel of truck equal to 12,000 lb (53.4 kN) for HS15 loading and 16,000 lb (71.1 kN) for HS20 loading (Chapter 10)	SN	= snow load
P	= load above ground (Chapter 11)	t	= actual time in days used in connection with shrinkage and creep (Chapter 5)
P_b	= design axial load strength of a section at simultaneous assumed ultimate strain of concrete and yielding of tension reinforcement (balanced conditions)	t	= age of concrete in days from loading (Chapter 5)
P_{cr}	= critical buckling load	t^*	= equivalent time in days used in connection with shrinkage (Chapter 5)
P_n	= nominal axial load at given eccentricity	t_w	= thickness of web in rectangular box section
P_{nx}	= nominal axial load at given eccentricity about x-axis	t_y	= temperature at distance y above depth of temperature variation of webs
P_{ny}	= nominal axial load at given eccentricity about y-axis	t_y'	= temperature reduction for asphalt concrete
P_{nxy}	= nominal axial load strength with biaxial loading	T	= temperature
P_o	= nominal load strength at zero eccentricity	T	= maximum temperature at upper surface of concrete (Chapter 5)
P_o	= at rest earth pressures (Chapter 5)	T	= fundamental period of vibration of the structure (Chapter 5)
P_s	= ratio of spiral reinforcement	T^*	= minimum temperature of top slab over closed interior cells (Chapter 5)
P_u	= moment, shear, or axial load from the i th loading (Chapter 5)	T_c	= nominal torsional moment strength provided by concrete
P_u	= factored axial load at given eccentricity, $P_u = \phi P_n$	T_n	= nominal torsional moment strength
P_{ux}	= factored axial load strength corresponding to M_{ux} with bending considered about the x-axis only	T_s	= nominal torsional moment strength provided by torsional reinforcement
P_{uy}	= factored axial load strength corresponding to M_{uy} with bending considered about the y-axis only	T_u	= factored torsional moment at section
P_{uxy}	= factored axial load strength with biaxial loading	v	= total applied design shear stress at section
q	= dynamic wind pressure	v_c	= permissible shear stress carried by concrete
		v_{dh}	= design horizontal shear stress at any cross section
		v_h	= permissible horizontal shear stress
		v_u	= factored shear stress at section
		V	= total applied design shear force at section

V	= horizontal earthquake force (Chapter 5)	y_1	= longer center-to-center dimension of closed rectangular stirrup
V	= velocity of water used in connection with stream flow (Chapter 5)	y_d	= mean thickness of deck between webs
V	= maximum probable wind velocity (Chapter 5)	y_t	= distance from the centroidal axis of cross section, neglecting the reinforcement, to the extreme fiber in tension
V_c	= nominal shear strength provided by concrete	Y_o	= depth of temperature variation of webs
V_{ci}	= nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment	Y_s	= height of temperature variation in soffit slab
V_{cw}	= nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web	z	= quantity limiting distribution of flexural reinforcement
V_i	= factored shear force at section due to externally applied loads occurring simultaneously with M_{max}	z	= height of top of superstructure above ground (Chapter 5)
V_n	= nominal shear strength provided by concrete and shear reinforcement	α	= angle between inclined shear reinforcement and longitudinal axis of member
V_{nh}	= nominal horizontal shear strength provided by concrete and shear reinforcement	α	= angle of pier inclination from vertical (Chapters 5 and 11)
V_p	= vertical component of effective prestress force at section considered	α	= load factor used in connection with group loadings (Chapter 5)
V_s	= nominal shear strength provided by shear reinforcement	α	= total angular change of prestressing steel profile (Chapter 9)
V_u	= factored shear force at section	α_v	= total vertical angular change of prestressing steel profile (Chapter 9)
w	= unit structure dead load	α_h	= total horizontal angular change of prestressing steel profile (Chapter 9)
w_c	= unit weight of concrete	α_f	= angle between shear friction reinforcement and shear plane
w_c	= roadway width between curbs (Chapters 10 and 11)	α_i	= load factor for the i th loading (Chapter 5)
w_e	= road slab width from edge of slab to midway between exterior beam and first interior beam	α_t	= factor used in connection with torsion reinforcement
W	= wind load used in connection with application of wind loads to different types of bridges	β	= percent of basic allowable stress (Chapter 5)
W	= total weight of structure (Chapter 5)	β_b	= ratio of area of bars cut off to total area of bars at section
W	= crack width (Chapter 11)	β_c	= ratio of long side to short side of concentrated load or reaction area
W_f	= gross weight of fatigue design truck	β_d	= ratio of maximum factored dead load moment to maximum factored total load moment, always positive
W_i	= gross weight of specific trucks used in determining fatigue design truck	β_1	= factor used to determine the stress block in ultimate load analysis and design
W_h	= wind load applied in horizontal plane	γ	= unit weight of soil
W_p	= weight of pier and footing below ground	δ_b	= moment magnification factor for braced frames
W_s	= weight of soil directly above footing	δ_s	= moment magnification factor for frames not braced against sidesway
W_v	= wind load applied in vertical plane	λ	= correction factor related to unit weight of concrete
WL	= wind load applied on live load (Chapter 5)	μ	= coefficient of friction
WL	= wind load on live load	μ	= curvature friction coefficient (Chapter 9)
x	= shorter overall dimension of rectangular part of cross section	μ	= ductility factor (Chapter 11)
x	= tandem spacing used in connection with aircraft loads (Chapter 5)	ξ	= time-dependent factor for sustained loads (Chapter 8)
x_1	= shorter center-to-center dimension of closed rectangular stirrup	$(\xi_{cr})_t$	= time-dependent factor for estimating creep under sustained loads (Chapter 5)
x_1	= distance from load to point of support (Chapter 10)	ξ_i	= instantaneous strain at application of load (Chapter 5)
x_2	= distance from center of post to point under investigation (Chapter 10)	$(\xi_{sh})_t$	= shrinkage at time t (Chapter 5)
y	= longer overall dimension of rectangular part of cross section	$(\xi_{sh})_u$	= ultimate shrinkage (Chapter 5)
y	= dual spacing used in connection with aircraft loads (Chapter 5)	ρ	= ratio of tension reinforcement A_s/bd
y	= height of box girder (Chapter 6)	ρ'	= ratio of compression reinforcement A'_s/bd

- ρ_b = reinforcement ratio producing balanced condition
 ρ_{min} = minimum tension reinforcement ratio = A_s/bd
 ρ_p = ratio of prestressed reinforcement = A_{ps}/bd
 ρ_s = ratio of volume of spiral reinforcement to total volume of core (out-to-out of spirals) of a spirally reinforced compression member
 ρ_v = $(A_s + A_h)/bd$
 ρ_w = reinforcement ratio = $A_s/b_w d$
 σ = moment magnification factor for compression members
 σ = effective ice strength (Chapter 5)
 τ_f = factor used in connection with prestressed concrete member design (Chapter 7)
 ϕ = strength-reduction factor
 ϕ = angle of internal friction (Chapter 5)

1.4—Referenced organizations

This report refers to many organizations which are responsible for developing standards and recommendations for concrete bridges. These organizations are commonly referred to by acronyms. Following is a listing of these organizations, their acronyms, full titles, and mailing addresses:

AASHTO

American Association of State Highway and Transportation Officials
 444 N. Capital Street, NW, Suite 225
 Washington, DC 20001

ACI

American Concrete Institute
 PO Box 9094
 Farmington Hills, MI 48333-9094

ANSI

American National Standards Institute
 1439 Broadway
 New York, NY 10018

AREA

American Railway Engineering Association
 50 F Street, NW
 Washington, DC 20001

ARTBA

American Road and Transportation Builders Association
 525 School Street, SW
 Washington, DC 20024

ASCE

American Society of Civil Engineers
 345 E. 47th Street
 New York, NY 10017

ASTM

ASTM International
 100 Barr Harbor Dr.
 West Conshohocken, PA 19428

AWS

American Welding Society
 550 NW LeJeune Road
 PO Box 351040
 Miami, FL 33135

BPR

Bureau of Public Roads
 This agency has been succeeded by the Federal Highway Administration

CEB

Comite European du Beton
 (European Concrete Committee)
 EPFL, Case Postale 88
 CH 1015 Lausanne
 Switzerland

CRSI

Concrete Reinforcing Steel Institute
 933 N. Plum Grove Road
 Schaumburg, IL 60195

CSA

Canadian Standards Association
 178 Rexdale Boulevard
 Rexdale (Toronto), Ontario
 Canada M9W 1R3

FAA

Federal Aviation Administration
 800 Independence Avenue, SW
 Washington, DC 20591

FHWA

Federal Highway Administration
 400 Seventh Street, SW
 Washington, DC 20590

GSA

General Services Administration
 18 F Street
 Washington, DC 20405

HRB

Highway Research Board
 This board has been succeeded by the Transportation Research Board

PCA

Portland Cement Association
 5420 Old Orchard Road
 Skokie, IL 60077

PCI

Precast/Prestressed Concrete Institute
 209 W. Jackson Blvd.
 Chicago, IL 60606-6938



PTI

Post-Tensioning Institute
301 W. Osborn, Suite 3500
Phoenix, AZ 85013

TRB

Transportation Research Board
National Research Council
2101 Constitution Avenue, NW
Washington, DC 20418

Recommended references

The documents of the various standards-producing organizations referred to in this report are listed below with their serial designation. The documents listed were the latest effort at the time this report was written. Since some of these documents are revised frequently, generally in minor detail only, the user of this report should check directly with the sponsoring group if it is desired to refer to the latest revision.

American Concrete Institute

104R	Preparation of Notation for Concrete
116R	Cement and Concrete Terminology





O'Hare Field elevated roadway (photo courtesy of Alfred Benesch and Company).

CHAPTER 2—REQUIREMENTS FOR BRIDGES

2.1—Introduction

2.1.1 General—Design of bridge structures should be in accord with requirements established by the owner, adapted to the geometric conditions of the site and in accord with the structural provisions of the applicable codes and specifications.

The geometry of the superstructure is dictated by the specified route alignment and the required clearances above and below the roadway. These requirements are in turn directly related to the type of traffic to be carried on the bridge deck, as well as that passing under the bridge and, when the site is near an airport, low flying aircraft. Thus, geometric requirements, in general, will be dependent on whether the bridge is to carry highway, railway, transit, or airplane traffic, and whether it is to cross over a navigable body of water, a highway, a railway, or a transit route. Drainage, lighting, and snow removal requirements should also be considered in the geometric design of the superstructure.

Once the overall geometry of the superstructure has been established, it should be designed to meet structural requirements. These should always include considerations of strength, serviceability, stability, fatigue, and durability. Before the reinforcing, prestressing, and concrete dimension requirements can be determined, an analysis should be performed to determine the internal forces and moments, the displacements, and the reactions due to the specified loadings on the bridge. This may be done using an elastic analysis, an empirical analysis, or a plastic model analysis as described in ACI SP-24. Because of their complexity, many bridge

structures have been analyzed by using an empirical approach. However, by coupling modern day analytical techniques with the use of digital computers, an elastic analysis of even the most complex structural systems can now be accomplished. Model analyses may prove useful when mathematical modeling is of doubtful accuracy, and especially in cases where a determination of inelastic and ultimate strength behavior is important.

2.1.2 Alignment—The horizontal and vertical alignment of a bridge should be governed by the geometrics of the roadway or channels above and below.

If the roadway or railway being supported on the bridge is on a curve, the most esthetic structure is generally one where the longitudinal elements are also curved. Box girders and slabs, if continuous, are readily designed and built on a curve. Stringers and girders can be curved but are more difficult to design and construct. If the curve is not sharp, the girders or stringers can be constructed in straight segments with the deck constructed on a curve. In this case the following points require close examination:

- a. Nonsymmetrical deck cross section.
- b. Deck finish of the “warped” surface.
- c. Vertical alignment of curbs and railing to preclude visible discontinuities.
- d. Proper development of superelevation.

Arches, cable-stayed, and suspension bridges are not easily adaptable to curved alignments.

2.1.3 Drainage—The transverse drainage of the roadway should be accomplished by providing a suitable crown or superelevation in the roadway surface, and the longitudinal drainage should be accomplished by camber or gradient. Water flowing downgrade in a gutter section of approach roadway should be intercepted and not permitted to run onto the bridge. Short continuous span bridges, particularly overpasses, may be built without drain inlets and the water from the bridge surface carried off the bridge and downslope by open or closed chutes near the end of the bridge structure. Special attention should be given to insure that water coming off the end of the bridge is directed away from the structure to avoid eroding the approach embankments. Such erosion has been a source of significant maintenance costs.

Longitudinal drainage on long bridges is accomplished by providing a longitudinal slope of the gutter (minimum of 0.5 percent preferred) and draining to scuppers or inlets which should be of a size and number to drain the gutters adequately. The positions of the scuppers may be determined by considering a spread of water of about one-half a lane width into the travel lane as recommended in "Drainage of Highway Pavements."²⁻¹ At a minimum, scuppers should be located on the uphill side of each roadway joint. Downspouts, where required, should be of rigid corrosion-resistant material not less than 4 in. (100 mm) and preferably 6 in. (150 mm) in the least dimension and should be designed to be easily cleaned. The details of deck drains and downspouts should be such as to prevent the discharge of drainage water against any portion of the structure and to prevent erosion at the outlet of the downspout.

Overhanging portions of concrete decks should be provided with a drip bead or notch within 6 in. (150 mm) of the outside edge.

2.2—Functional considerations

2.2.1 Highway bridges

2.2.1.1 Highway classification—Highways are classified by types for their planning, design, and administration. The classification in each jurisdiction is made in accordance with the importance of the highway, the traffic volume, the design speed, and other pertinent aspects. The following functional considerations are dependent upon the highway classifications:

2.2.1.2 Width—The roadway width (curb-to-curb, rail-to-rail, or parapet-to-parapet distance) is dependent on the number of traffic lanes, the median width, and the shoulder width. The preferred roadway width should be at least that distance between approach guardrails, where guardrails are provided, or the out-to-out approach roadway, and shoulder width as recommended in AASHTO HB-12. Reduced widths are sometimes permitted where structure costs are unusually high or traffic volumes unusually low. Where curbed roadway sections approach a structure, the same section should be carried across the structure.

Recommendations as to roadway widths for various volumes of traffic are given in AASHTO DS-2, DSOF-3, GD-2 and GU-2.

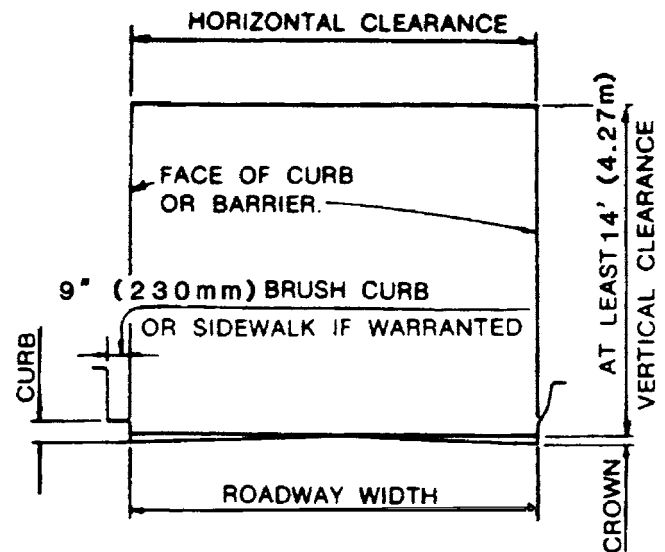


Fig. 2.2.1.3—Clearance diagram for bridges.

2.2.1.3 Clearances—The horizontal vehicular clearance should be the clear width measured between curbs or sidewalks, and the vertical clearance should be the clear height for the passage of vehicular traffic measured above the roadway at the crown or high point of superelevation (Fig. 2.2.1.3).

Unless otherwise provided, the several parts of the structure should be constructed to secure the following limiting dimensions or clearances for traffic:

- The minimum horizontal clearance for low traffic speed and low traffic volume bridges should be 8 ft (2.4 m) greater than the approach travelled way. The clearance should be increased as speed, type, and volume of traffic dictate in accordance with AASHTO DS-2, DSOF-3, GD-2, and GU-2.
- Vertical clearance on state trunk highways and interstate systems in rural areas should be at least 16 ft (5 m) over the entire roadway width, to which an allowance should be added for resurfacing. On state trunk highways and interstate routes through urban areas, a 16-ft (5-m) clearance should be provided except in highly developed areas. A 16-ft (5-m) clearance should be provided in both rural and urban areas, where such clearance is not unreasonably costly and where needed for defense requirements. Vertical clearance on all other highways should be at least 14 ft (4.25 m) over the entire roadway width to which an allowance should be added for resurfacing.

2.2.1.4 Sidewalks—Sidewalks, when used on bridges, should be as wide as required by the controlling and concerned public agencies, and preferably should be 5 ft wide (1.5 m) but not less than 4 ft (1.25 m).

2.2.1.5 Curbs—There are two general classes of curbs. These are "parapet" (nonmountable) and "vehicular mountable" curbs. Both may be designed with a gutter to form a combination curb and gutter section. The minimum width of curbs should be 9 in. (225 mm). Parapet curbs are relatively high and steep faced. They should be designed to prevent the vehicle from leaving the roadway. Their height varies, but it

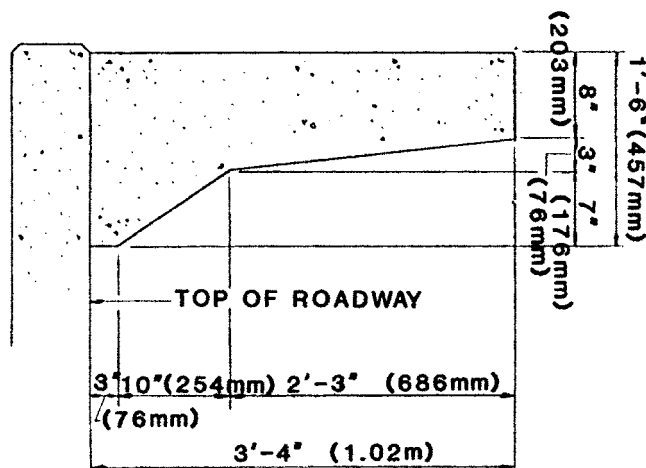


Fig. 2.2.1.5—Parapet curb and railing section.

should be at least 2 ft-3 in. (700 mm). When used with a combination of curb and handrail, the height of the curb may be reduced. Fig. 2.2.1.5 shows a parapet curb and railing section which has demonstrated superior safety aspects, and is presently used by state highway offices. Mountable curbs, normally lower than 6 in. (150 mm), should not be used on bridges except in special circumstances when they are used in combination with sidewalks or median strips. The railing and curb requirements, and the respective design loads, are indicated in AASHTO HB-12. Curbs and sidewalks may have vertical slits or other provisions for discontinuity, to prevent them from participating in deck bending moments, to reduce cracking of these elements.

2.2.1.6 Medians—On major highways the opposing traffic flows should be separated by median strips. Wherever possible, the lanes carrying opposing flows should be separated completely into two distinct structures. However, where width limitations force the utilization of traffic separators (less than 4 ft wide) the following median sections should be used:

- Parapet sections 12 to 27 in. (300 to 700 mm) in height, either integral or with a rail section, are recommended in "Location, Section, and Maintenance of Highway Traffic Barriers."²⁻² The bridge and approach parapets should have the same section.
- Low rolled curb sections or double curb units with some form of paved surface in between are recommended for low-speed roads in "Handbook of Highway Safety Design and Operating Practices."²⁻³

2.2.1.7 Railing—Railing should be provided at the edge of the deck for the protection of traffic or pedestrians, or both. Where pedestrian walkways are provided adjacent to roadways, a traffic railing may be provided between the two, with a pedestrian railing outside. Alternatively, a combination traffic-pedestrian railing may be used at the outside of the pedestrian walkway. Railings may be made of concrete, metal, timber or a combination of these materials. The service loads for the design of traffic and pedestrian railings are specified in AASHTO HB-12.

While the primary purpose of traffic railing is to contain the average vehicle using the structure, consideration should

also be given to protection of the occupants of a vehicle in collision with the railing, to protection of other vehicles near the collision, to vehicles or pedestrians on roadways being overcrossed, and to appearance and freedom of view from passing vehicles. Traffic railings should be designed to provide a smooth, continuous face of rail. Structural continuity in the rail members (including anchorage of ends) is essential. The height of traffic railing should be no less than 2 ft-3 in. (700 mm) from the top of the roadway, or curb, to the top of the upper rail members. Careful attention should be given to the treatment of railing at the bridge ends. Exposed rail ends and sharp changes in the geometry of the railing should be avoided. The approach end of all guardrail installations should be given special consideration to minimize the hazard to the motorist. One method is to taper the guardrail end off vertically away from the roadway so that the end is buried as recommended in "Handbook of Highway Safety Design and Operating Practices."²⁻³

Railing components should be proportioned commensurate with the type and volume of anticipated pedestrian traffic, taking account of appearance, safety, and freedom of view from passing vehicles. The minimum design for pedestrian railing should be simultaneous loads of 50 lb/ft (730 N/m) acting horizontally and vertically on each longitudinal member. Posts should be designed for a horizontal load of 50 lb (225 N) times the distance between posts, acting at the center of gravity of the upper rail.

The minimum height of pedestrian railing should be 3 ft-6 in. (1.1 m), measured from the top of the walkway to the top of the upper rail member. Railings for walkways that are also used as bicycle paths should have a height of 4 ft-6 in. (1.4 m).

2.2.1.8 Superelevation—Superelevation of the surface of a bridge on a horizontal curve should be provided in accordance with the applicable standard for the highway. The superelevation should preferably not exceed 6 percent, and never exceed 8 percent.

2.2.1.9 Surfacing—The road surface should be constructed following recommendations in ACI 345.

2.2.1.10 Expansion joints—To provide for expansion and contraction, joints should be provided at the expansion ends of spans and at other points where they may be desirable. In humid climates and areas where freezing occurs, joints should be sealed to prevent erosion and filling with debris, or else open joints should be properly designed for the disposal of water.

A State-of-the-Art Report on Joint Sealants is given in ACI 504R.

2.2.2 Railway bridges

2.2.2.1 Railway classification—Rail lines are classified by their purpose and function. Each type has its own requirements for design, construction, and maintenance.

2.2.2.2 Width—The width of the bridge should be based on the clearance requirements of *AREA Manual for Railway Engineering*, Chapter 28, Part 1, or to the standards of the railway having jurisdiction.

2.2.2.3 Clearances—Minimum clearances should be in accordance with the requirements of the railway having

jurisdiction. Minimum clearances established by AREA are indicated in Fig. 2.2.2.3.

2.2.2.4 Deck and waterproofing—All concrete decks supporting a ballasted roadbed should be adequately drained and waterproofed. The waterproofing should be in accordance with the provisions outlined in *AREA Manual of Railway Engineering*, Chapter 29.

2.2.2.5 Expansion joints—To provide for expansion and contraction movement, deck expansion joints should be provided at all expansion ends of spans and at other points where they may be necessary. Apron plates, when used, should be designed to span the joint and to prevent the accumulation of debris on the bridge seats. When a waterproof membrane is used, the detail should preclude the penetration of water onto the expansion joint and bridge seat.

2.2.3 Aircraft runway bridges—The runway width, length, clearances, and other requirements should conform to the provisions of the Federal Aviation Agency or other air service agency having jurisdiction.

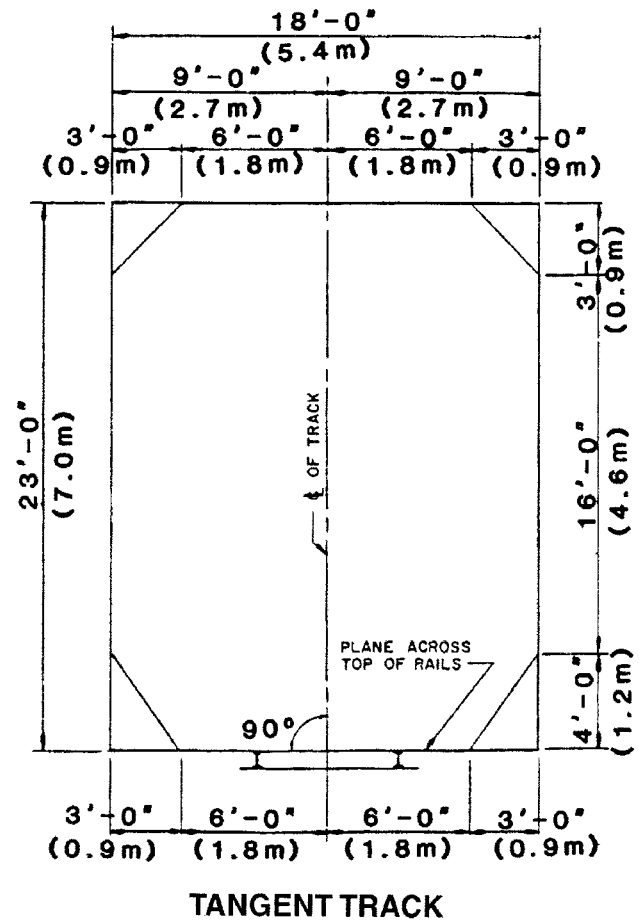
2.2.4 Transit bridges—A transit bridge or guideway differs from a conventional highway bridge in that it both supports and guides an independent transit vehicle. Special considerations are required in the design and construction to attain the desired level of ride comfort. A State-of-the-Art report for concrete guideways is given in ACI 358R.

2.2.5 Spans and profile

2.2.5.1 General—In addition to providing the proper surface to carry the proposed traffic on the bridge, the structure should provide proper clearance for the facility being crossed.

2.2.5.2 Stream and flood plain crossings—The bridge should be long enough to provide the required waterway opening below the high water elevation of the design flood. The opening provided should be an effective opening, i.e., be measured at a right angle to the stream centerline, furnish adequate net opening, have adequate upstream and downstream transitional cleanouts, be vertically positioned between the stable flowline elevation and the correct frequency highwater elevation, and be positioned horizontally to most efficiently pass the design volume of water at the design flood stage. Provision should be made for foreseeable natural changes in channel location and, if necessary, channel realignment should be made a part of the bridge construction project.

The bridge waterway opening and the roadway profile together determine the adequacy of the system to pass floods. The roadway and stream alignments determine the effectiveness of the provided opening and they influence the need for spur dikes, erosion protection, structure skew, and pier locations. Detailed guidelines are given in AASHTO HDG-7. Good practice usually dictates that the lowest elevation of the superstructure should not be lower than the all-time record high water in the vicinity of the crossing and that an appropriate clearance be provided above the design high-water elevation. The amount of clearance depends on the type of debris that should pass under the bridge during floods and the type of bridge superstructure. When costs of meeting this requirement are excessive, consideration should be given to other means of accommodating the unusual floods,



a) The clearances shown are for tangent track and new construction. Clearances for reconstruction work or for alteration are dependent on existing physical conditions and, where reasonably possible, should be improved to meet the requirements for new construction.

b) On curved track, the lateral clearances each side of track centerline should be increased 1-1/2 in. (38 mm) per deg of curvature. When the fixed obstruction is on tangent track but the track is curved within 80 ft (24 m) of the obstruction, the lateral clearances each side of track centerline should be as follows:

Distance from obstruction to curved track, ft		Increase per deg of curvature, in.	
Feet	(Meters)	Inches	(Millimeters)
20	(6)	1-1/2	(38)
40	(12)	1-1/8	(28)
60	(18)	3/4	(19)
80	(24)	3/8	(9.5)

c) On the superelevated track, the track centerline remains perpendicular to a plane across top of rails. The superelevation of the outer rail should be in accordance with the recommended practice of the AREA.

d) In some instances, state or Canadian laws and individual railroads require greater clearances than these recommended minimums. Any facility adjacent to, or crossing over, railroad tracks should not violate applicable state laws, Canadian law, or requirements of railroads using the tracks.

Fig. 2.2.2.3—Clearance diagram for railroads.

such as lowering the approach embankment to permit overtopping. If any part of the bridge superstructure is below the all-time record high water, it should be designed to resist flow pressures and anchored accordingly.

Requirements for stream crossings should be obtained from the governmental agency having responsibility for the stream being crossed. For further recommendations, see *Hydraulic Design of Bridges with Risk Analysis*.²⁻⁴

2.2.5.3 Navigable stream crossings—Vertical clearance requirements over navigation channels vary from 15 to 220 ft (4.5 to 67 m) and are measured above an elevation determined by the U.S. Coast Guard, or in Canada by Transport Canada. Horizontal clearances and the location of the opening depend upon the alignment of the stream upstream and downstream of the bridge. In some cases auxiliary navigation channels are required.

2.2.5.4 Highway crossings (Fig. 2.2.5.4)—The pier columns or walls for grade separation structures should generally be located a minimum of 30 ft (9 m) from the edges of the through traffic lanes. Where the practical limits of structure costs, type of structure, volume and design speed of through traffic, span arrangement, skew, and terrain make the 30-ft (9-m) offset impractical, the pier or wall may be placed closer than 30 ft (9 m) and protected by the use of guard rail or other barrier devices. The guard rail should be independently supported with the roadway face at least 2 ft (600 mm) from the face of pier or abutment.

The face of the guard rail or other device should be at least 2 ft (600 mm) outside the normal shoulder line.

A vertical clearance of not less than 14 ft (4.25 m) should be provided between curbs, or if curbs are not used, over the entire width that is available for traffic. Curbs, if used, should match those of the approach roadway section.

2.2.5.5 Railway crossings (Fig. 2.2.5.5)—In addition to the requirements shown in Fig. 2.2.5.5, it is good practice to allow at least 6 in. (150 mm) for future track raise. In many instances the railroad requires additional horizontal and vertical clearance for operation of off-track equipment. Piers located closer than 25 ft (7.5 m) from the track should meet the requirements of *AREA Manual for Railway Engineering*, Chapter 8, Subsection 2.1.5, to be of heavy construction or to be protected by a reinforced concrete crash wall extending 6 ft (2 m) above top of rail. In certain instances where piers are adjacent to main line tracks, individual railways may have more stringent requirements.

2.3—Esthetic considerations

A bridge should be designed to harmonize with its natural surroundings and neighboring structures. The attractiveness of a bridge is generally achieved by its shape and by the proper proportioning of the superstructure and piers in relation to the span of the bridge and its surroundings. Color and texture may be added for emphasis. Consideration should be given to the appearance of the bridge from the driver's or passenger's point of view, as well as someone viewing it from off the structure. A bibliography of books and articles on bridge esthetics has been published (Reference 2-5) and is available from the Transportation Research Board.

2.4—Economic considerations

2.4.1 Criteria for least cost—Least-cost criteria require consideration of all the factors contributing to the cost of the

project. These include length and width of superstructure, type of superstructure including deck, railings, walks, medians; type of substructure including cofferdams, sheeting, and bracing, approach roadways including embankment, retaining walls and slope protection. Other factors such as special treatment for the road or stream being spanned, and pier protection, can also influence the least cost.

Each type of superstructure being considered has an optimum span range where its use is very competitive. It may, however, be used in spans outside that range and still meet the least-cost criteria, because of the compensating costs of other factors. One of the compensating factors often is the substructure because its contribution to the cost of the project is inversely proportional to the span length, while the superstructure cost increases with the span length. Wherever possible, consideration should be given to comparing bridge layouts having different span arrangements. Elimination of a costly river pier can usually justify a longer span.

Although this report stresses the design of the superstructure, the substructure of any bridge is a major component of its cost and for that reason offers an almost equally great potential for cost saving.

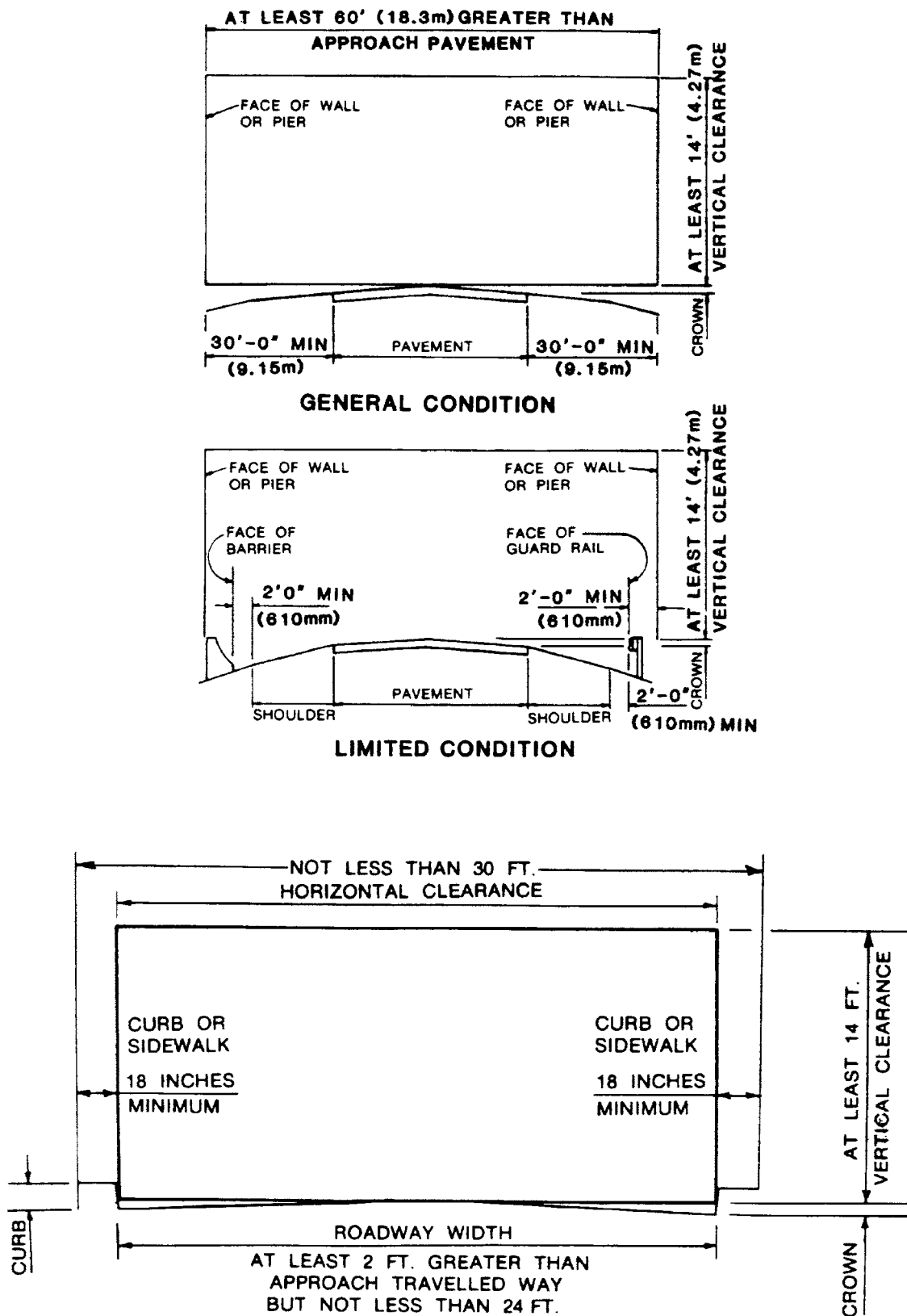
In the final analysis, however, true economy is measured by the minimum annual cost or minimum capitalized cost for its service life. Cost data on maintenance, repair or rehabilitation, and estimate of useful life are less easy to obtain, but no study of least cost can be complete without their consideration. For cost of bridges within flood plains, see *Hydraulic Design of Bridges with Risk Analysis*.²⁻⁴

2.4.2 Alternative designs—The general statement that a competent engineer can establish the most economical structure by studies ignores factors which influence costs over which the engineer has no control. The economics of any given industry cannot be exactly forecast. The time of advertising most structures is not established at the time of design. The reasons for preparing alternative designs are:

- Increase competition by permitting several industries to participate.
- Make provisions to take advantage of the variations in the economy of the construction industry.
- To provide a yardstick whereby the various industries can measure the advantage and disadvantage of their competitive position. This results in industry improving their procedures to reduce costs and eventually gives additional savings to the owner.
- To eliminate the intangible arguments by various segments of industry that their material would have resulted in a more economical structure.
- Most important reason for alternative designs is that the owner saves in the cost of the structure.

For more detailed information consult *Alternate Bridge Designs*.²⁻⁶

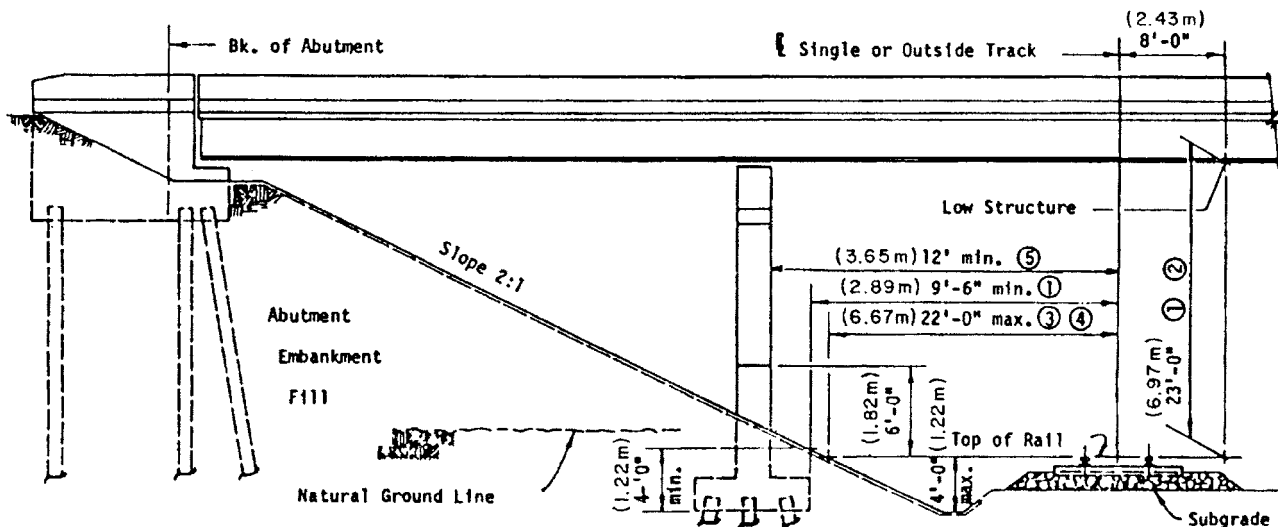
2.4.3 Value engineering—In addition to economic pressures, sociological pressures have focused more attention on the impact that a project has on both natural and cultural environments. Consequently, the bridge engineer is faced with the necessity of identifying a continually growing list of design parameters, along with the accompanying possibility



1) For recommendations as to roadway widths for various volumes of traffic, see AASHTO DS-2, DSOF-3, GD-2, and GU-2.

2) The barrier to face of wall or pier distance should not be less than the dynamic deflection of the barriers for impact by a full-size automobile at impact conditions of approximately 25 deg (0.44 rad.) and 60 mph (96.5 km/h). For information on dynamic deflection of various barriers, see AASHTO STB.

Fig. 2.2.5.4—Clearance diagrams for underpasses.



**TYPICAL SECTION AT RIGHT ANGLE TO TRACK
NATURAL GROUND LESS THAN 4'-0" (1.22m) BELOW T/R**

HIGHWAY - RAILROAD OVERHEAD GRADE SEPARATIONS NOMINAL DESIGN DIMENSIONS

Notes:

- 1 Do not reduce without the consent of the Railroad Company.
- 2 Do not reduce below 21'-6" (6.5m) without an I.C.C. Order.
- 3 This dimension may be increased up to 8'-0" (2.43m), on one side only, as may be necessary for off-track maintenance equipment when justified by Railroad Company.
- 4 This dimension may be increased up to 3'-0" (0.91m), where special conditions, such as heavy and drifting snow, are a problem.
- 5 Piers or columns are to be located so as not to encroach on drainage ditches.

All horizontal dimensions are at right angles.

Fig. 2.2.5.5—Clearances for railway crossings.

of tradeoffs in the process of planning and designing. Selection of not only a suitable type of substructure and superstructure, but a suitable location with the consideration of all these factors, can be very complex. A complete and objective result can be accomplished only if an organized approach is adopted. Value Engineering is one such system that can help engineers obtain an optimum value for a project.

Value Engineering is an organized way of defining a problem and creatively solving it. The Value Engineering Job Plan has five steps: 1) information phase, 2) analysis phase, 3) speculative phase, 4) evaluation phase, and 5) implementation phase. The Job Plan encourages engineers to search systematically, analyze objectively, and solve creatively. Details of the Value Engineering method can be

obtained from *Guidelines for Value Engineering*²⁻⁷ or *Value Engineering for Highways*.²⁻⁸

Value Engineering can be used at various stages of a project, but the earlier the process is initiated, the greater the possible benefits. This is graphically illustrated by Fig. 2.4.3, taken from the book *Value Engineering in Construction*.²⁻⁹ Value Engineering when used only in the Value Engineering Change Proposal (VECP) produces limited benefits because only the low bidder on the "owner's design" is permitted to submit a Value Engineering alternate. Its use is discussed in Section 4.12 of this report.

2.5—Bridge types

Bridges may be categorized by the relative location of the main structural elements to the surface on which the users

travel, by the continuity or noncontinuity of the main elements and by the type of the main elements.

2.5.1 Deck, half-through, and through types (see Fig. 2.5.1)—To insure pedestrian safety, bridges designed with sidewalks should preferably permit an unobstructed view. This requirement is satisfied with deck bridges where the load-carrying elements of the superstructure are located entirely below the traveled surface.

In rare cases, clearances may justify a half-through or through-type structure when the difference between the bridge deck elevation and the required clearance elevation is small. In through-type structures, the main load-carrying elements of the superstructure project above the traveled surface a sufficient distance such that bracing of the main load-carrying element can extend across the bridge. Thus the bridge user passes “through” the superstructure.

In half-through structures, the main carrying elements are braced by members attached to and cantilevering from the deck framing system. The top flanges of half-through girders or top chords of half-through trusses are much less stable than those in deck and through structures. In the deck structure in particular, the roadway slab serves very effectively to increase the lateral rigidity of the bridge. The projecting elements of the half-through or through structures are very susceptible to damage from vehicles and adequate protection should be provided. These types also do not permit ready widening of the deck in the future.

2.5.2 Simple, cantilever, and continuous span types (see Fig. 2.5.2)—Concrete bridges may consist of simple, cantilever, or continuous spans. Continuous structures with variable moments of inertia for slabs, stringers, and girder systems require the least material. However, in the shorter span range the labor cost of constructing variable sections often offsets the material savings.

In the past the use of cantilever arms and suspended spans rather than continuous structures resulted from fear of the effect of differential settlement of supports. It should be recognized that these effects can now be readily considered by proper use of analytical methods and current knowledge in soil mechanics and foundation engineering. Because of the difficult problems of detailing and constructing the bearing which forms the hinge at the end of the cantilever, its use should be limited to special situations. It is also inadvisable to use hinges in areas subject to seismic loadings.

In the longer span range of slab, stringer, and girder-type bridges, the use of continuous structures with variable moments of inertia is strongly recommended. It should also be recognized that the designer may sometimes take advantage of the economy of a combination of bridge types. Short approach spans of slab or box beam construction may be combined with a single main arch span or with long box girder spans.

2.5.3 Slab, stringer, and girder types—These types may be either square or skew in plan and simply supported, cantilevered, or continuous over supports.

2.5.3.1 Slab type (see Fig. 2.5.3.1)—Slab-type concrete bridges consist of solid or voided slabs which span between abutments and intermediate piers. The use of slab bridges should be considered for spans up to 80 ft (25 m). The main

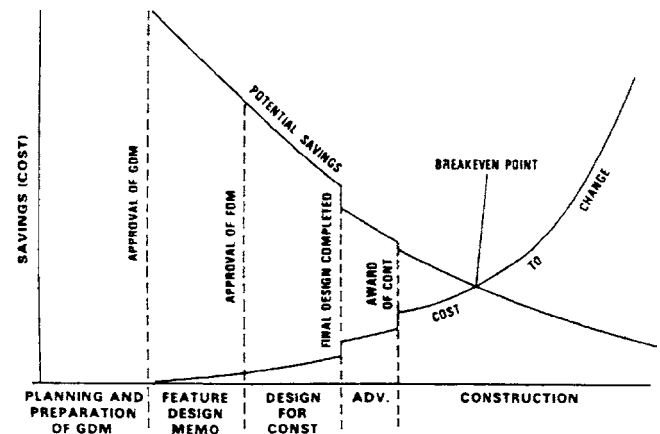


Fig. 2.4.3—Potential Value Engineering savings during civil works project life cycle.

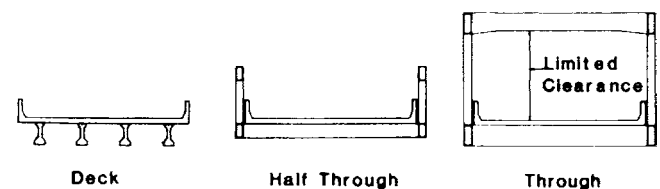


Fig. 2.5.1—Cross sections of bridge types.

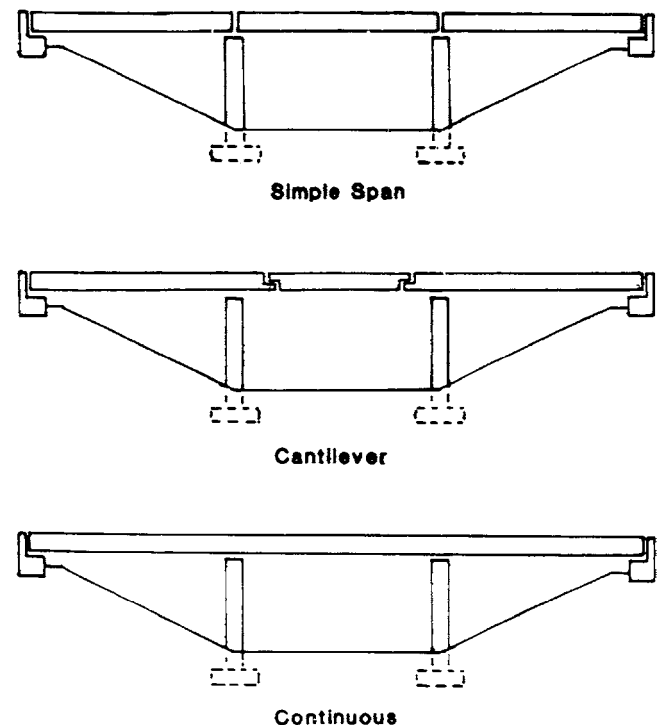


Fig. 2.5.2—Elevations of bridge types.

longitudinal reinforcement may be prestressed or nonprestressed. Transverse prestressing can be used for transverse reinforcement in both cast-in-place slabs and those composed of longitudinal segments.

2.5.3.2 Stringer type (see Fig. 2.5.3.2)—The main structural elements of this type of concrete bridge consist of a



Fig. 2.5.3.1—Cross sections of slab bridges.



Fig. 2.5.3.2—Cross sections of stringer bridges.

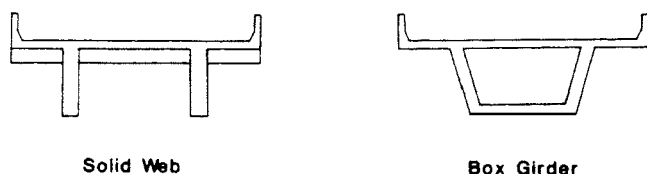


Fig. 2.5.3.3—Cross sections of girder bridges.

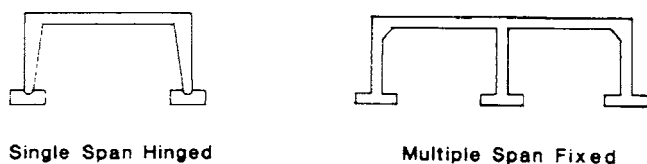


Fig. 2.5.4—Elevations of rigid frame bridges.

series of parallel beams or stringers which may or may not be connected with diaphragms. The stringers support a reinforced concrete roadway slab which is generally constructed to act as the top flange of the stringer. Use of stringer bridges should be considered for spans ranging from about 20 to 120 ft (6 to 36 m). Stringers generally are spaced from 6 to 9 ft (1.8 to 2.7 m) on centers. Main reinforcement or prestressing is located in the stringers. When the stringers are prestressed, the concrete roadway slab may be reinforced or prestressed in two directions.

2.5.3.3 Girder type (see Fig. 2.5.3.3)—This type of concrete bridge consists of either longitudinal girders carrying cross beams that in turn carry the roadway slab, or longitudinal box girders whose bottom slab functions as the bottom flange of the girder. In both types the top slab serves the dual function of being the flange of the girder and the roadway slab of the bridge.

Solid web girders—This type of bridge differs from the stringer type in that the reinforced concrete roadway slab spans longitudinally and is supported on cross beams spaced 8 to 12 ft apart (2.4 to 3.6 m). In general only two girders are used. It is a feasible solution where large overhangs of the deck are desirable, and where the depth of construction is not critical. Spans of up to 180 ft (55 m) have been built.

Box girders—These girders may consist of a single cell for a two-lane roadway, multiple cells for multiple-lane roadways, or single or multiple cells with cantilever arms on both sides to

provide the necessary roadway width, and to reduce the substructure cost and minimize right of way requirements.

Because of their superior torsional rigidity, box girders are especially recommended for use on curves or where torsional shearing stresses may be developed. This type of concrete bridge may have economical span lengths from less than 100 to about 700 ft (30 to 210 m). In the longer span range, variable depth, variable moment of inertia, cantilever construction, continuity design, prestressed precast elements, precast post-tensioned components, post-tensioned construction, three-way prestressing for the whole bridge assembly, and introduction of continuity after erection should all be considered.

2.5.4 Rigid-frame type (see Fig. 2.5.4)—Rigid-frame bridges may be hinged or fixed, single or multiple spans. The main structural elements are generally slabs, but may be beams. Depending upon the roadway planning, each span may accommodate from one lane with shoulders to as many as four lanes with shoulders. A two-span structure can accommodate roadways in each direction with a narrow median. A three-span structure can accommodate roadways in each direction with a wide median. Overhangs may be introduced to span over embankment slopes and to reduce the moment at the knees of the first legs of a rigid-frame system. The base of legs or columns may be either hinged or fixed. In either case, the hinge action or fixity should be constructed to fit the design conditions. Feasible span lengths are similar to slab or stringer-type bridges.

The bridge may be square or skew to suit the geometric or hydraulic requirements. Where possible, large skews should be avoided by changing the alignment of the supported roadway or the obstruction being crossed. If this cannot be done, a detailed analysis should be used in design of the structure. In restricted locations, barrel-type bridges may be used to dispense with abutments. Both composite construction and two-way reinforcing or two-way prestressing may be applied to the slab-rigid-frame assembly.

2.5.5 Arch type—Arches generate large horizontal thrusts at their abutments, and are therefore ideally suited for crossings of deep gorges or ravines whose rock walls provide a relatively unyielding support. If founded on less suitable material, the deformation of the foundation should be taken into account in the analysis. Arches are also best suited for structures that have a sizable dead load-to-live load ratio. Arch construction may be made of cast-in-place or precast segmental elements, as well as being conventionally formed.

2.5.5.1 Spandrel or barrel arches (see Fig. 2.5.5.1)—These arches are often fixed at the springing lines on the abutments. Two-hinged and three-hinged arches are seldom used. More often, single spans are used. In this type of bridge, the spandrels act as retaining walls for earth fill which is placed on top of the arch to form the subbase for the roadway. It has distinct advantages for short spans, low rises, and heavy live loads. Spans up to 200 ft (60 m) have been used. However, in spans over 100 ft (30 m), the dead load due to the earth fill may become excessive.

2.5.5.2 Ribbed or open-spandrel arches (see Fig. 2.5.5.2)—This type of bridge is suitable for long spans. Both

fixed and two-hinged arches are common. The three-hinged version is feasible but is seldom used.

Ribbed arches are better adapted to multiple spans than barrel arches. The roadway deck is supported on columns and cross beams. It may be a deck structure or half-through structure. In the latter case, hangers should be used to carry the roadway in the central portion of the arch. In this type of bridge, these are usually two separate arch ribs, but there may be more or the rib may be a solid slab, in which case the columns and cross beams may be replaced by walls. Spans up to 1000 ft (300 m) have been built.

2.5.5.3 Tied arches (see Fig. 2.5.5.3)—This type of arch should be used where the foundation material is considered inadequate to resist the arch thrust. Tied arches are always of the through or half-through type with hangers to carry floor beams. Both single-span and multiple-span tied arch bridges have been built with spans of 100 to 400 ft (30 to 120 m).

2.5.5.4 Long-span arches—Ribbed arches with precast box segments, post-tensioned during assembly, are suitable for long spans. In these bridges, post-tensioned diaphragms have been used.

2.5.5.5 Splayed arches or space frame—To achieve greater stability and structural stiffness, through-type arches may be inclined to each other to form a space frame. This results in a splayed arch system.

For shorter spans, vertical cable suspenders may be used. In the longer span range, diagonal-grid cable suspenders are recommended for attaining greater stiffness.

The diagonal-grid concept may be further extended to the floor system by having a grid of beams running diagonally to the center line of the roadway.

2.5.6 Truss types—Although concrete truss bridges of triangular configuration have been built, their use is not recommended. The detail of reinforcement at a joint where many members meet is very difficult. Formwork, centering, and placing of concrete in sloping members is expensive.

2.5.6.1 Vierendeel truss (see Fig. 2.5.6.1)—While not used extensively, the Vierendeel truss offers some esthetic qualities, has simpler details because of the limited number of members at a joint, is easier to form and place, and can be precast or cast in place. If necessary, it can be erected by cantilever method without false work.

Previously, a deterrent to the use of the Vierendeel truss was the difficulty in analyzing this highly indeterminate structure, but computer programs are now available to rapidly do this analysis.

The use of inclined chords, particularly in the end panel, is recommended. This inclination greatly reduces the bending stresses.

Because of the great depth needed for efficient use, the Vierendeel truss bridge is generally either a through or half-through type. It is also best suited for simple spans. Spans up to 500 ft (150 m) have been built.

2.5.7 Cable-stayed types (see Fig. 2.5.7)—The feasible span of concrete bridges can be greatly extended by the use of supplementary supports. In cable-stayed bridges the concrete deck, including the roadway slab and girders, acts as a part of the support system, functioning as a horizontal

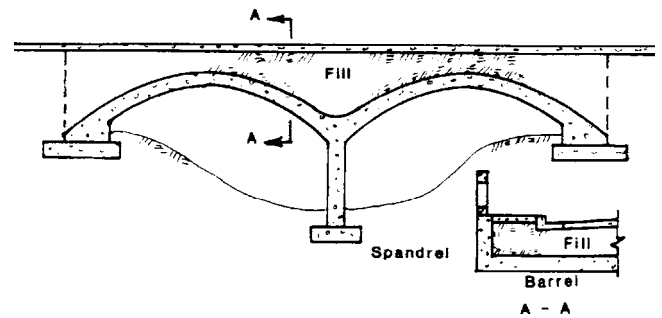


Fig. 2.5.5.1—Two span spandrel or barrel (earth filled) arch bridge.

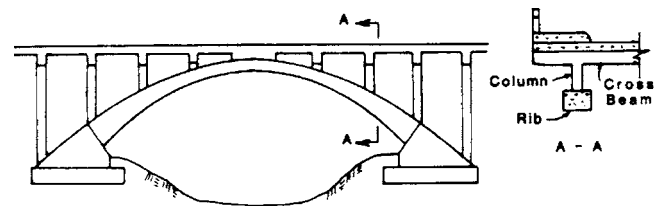


Fig. 2.5.5.2—Single span open spandrel or ribbed arch bridge.

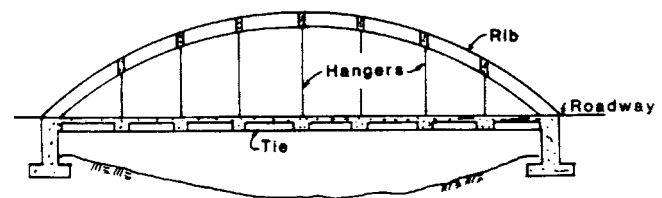


Fig. 2.5.5.3—Tied arch bridge.

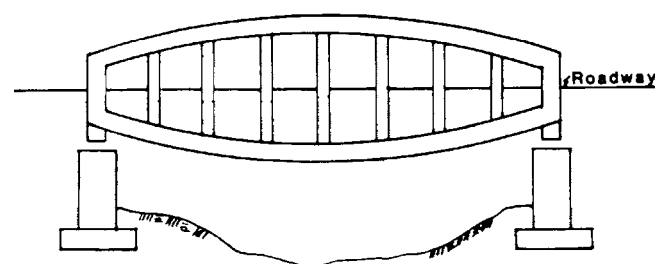


Fig. 2.5.6.1—Vierendeel truss bridge.

compression member. Cable-stayed bridges act as continuous girders on flexible supports, and offer the advantage of high rigidity and aerodynamic stability with a low level of secondary stresses. Spans of up to 1500 ft (460 m) are feasible.

The proper design of the pylon and configuration of the stays can add greatly to the esthetic appearance.

2.5.8 Suspension types (see Fig. 2.5.8)—In suspension bridges, intermediate vertical support is furnished by hangers from a pair of large cables. The deck does not participate except to span between hangers and to resist horizontal loads.

Because dead load is a very important factor, high-strength lightweight concrete should be considered. Spans of 1600 to 1800 ft (489 to 550 m) have been built.

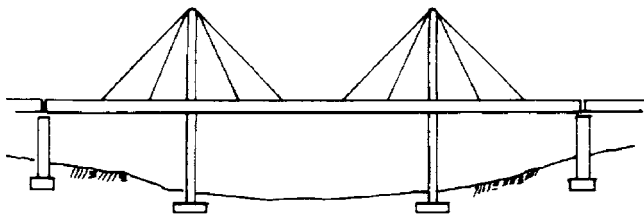


Fig. 2.5.7—Cable stayed bridge.

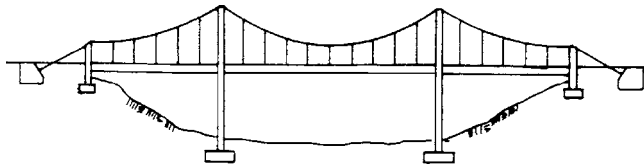


Fig. 2.5.8—Suspension bridge.

2.6—Construction and erection considerations

In the design of a bridge, construction and erection considerations may be of paramount importance in the selection of the type of bridge to be built. Also, the experience of the available contractors, the ability of local material suppliers to furnish the specified materials, the skilled labor required for a particular structure type, and the capacity of equipment necessary for erection should be considered. The most economical bridge design is one in which the total cost of materials, labor, equipment, and maintenance is minimized.

2.6.1 Cast-in-place and precast concrete—The decision to use or not to use precast concrete could be influenced by the availability of existing precast plants within transport distance. Precast concrete may be competitive in areas without existing precasting plants when a large number of similar components are required.

In large projects, a precasting plant located at the site should be considered to see if it would prove more economical.

In general, precast concrete members, because of better control of casting and curing processes, and because of the ease of inspection and rejection of an improperly fabricated member, are a better, more durable product.

For grade separation structures, if traffic problems are not a controlling factor, cast-in-place structures are generally more economical when the height of falsework is less than 30 to 40 ft (9 to 12 m) high.

2.6.2 Reinforced, partially prestressed, and prestressed—This report covers use of reinforced concrete, partially prestressed concrete, and fully prestressed concrete. The possible use of pretensioning or post-tensioning should be considered during the planning stage of a bridge project. In many cases the greatest economy can be realized by allowing the Contractor the option of using pretensioned, post-tensioned, or a combination of both. In these cases, the specifications should require submittal by the Contractor of proper design data.

2.6.3 Composite construction—Integration of the deck slab with the supporting floor system is covered by this document. Floor systems consisting of stringers, floor beams, or combinations can be used. Modular precast

concrete planks (prestressed or regular reinforced) may be used as the bottom form for the deck slab between stringers. Properly designed, these planks can be made composite with the cast-in-place deck slab and the deck slab composite with the stringers. Consideration should be given in the design to construction loads supported prior to the cast-in-place concrete attaining its design strength. For short spans within the capacity of available handling equipment, the entire deck span may be precast in one piece and made composite with the cast-in-place slab.

2.6.4 Post-tensioned segmental construction—It is normal practice to build concrete bridges in segments such as precast I-beams with composite slabs or precast voided slabs or box beams that are attached together. In the post-tensioned segmental type, the individual member, box girder, I-beam, or arch is installed in several longitudinal segments and then post-tensioned together to form one member.

2.6.4.1 Box girders—In general, the longer spans, because of the need for greater and variable depths, have been cast-in-place, while the shorter spans lend themselves to constant depth precast units. It is customary to erect these bridges by the cantilever method, avoiding the use of falsework, but some have been erected using a limited amount of falsework and placing the bridge by “pushing” the completed segments into place from one end.

2.6.4.2 I-beams—Due to shipping limitations, the length of precast prestressed I-beam stringer bridges is less than 100 ft (30 m). By precasting the I-beam in two or more pieces and post-tensioning the pieces after erection, the feasible span can be greatly increased.

2.6.4.3 Arches—Arches of all types may be constructed of cast-in-place or precast segments. This method of construction is most adaptable to long spans and spans where centering for formwork is difficult to install. After constructing the arch ribs by the segmental method, the spandrel columns or suspenders and the roadway deck may be constructed in a more conventional manner.

2.7—Legal considerations

2.7.1 Permits over navigable waterways—Preliminary plans of a proposed bridge crossing any navigable waterway should be filed with the Commandant, U.S. Coast Guard, addressed to the appropriate District Commander, and with other appropriate Governmental authority. A written permit with reference to horizontal and vertical clearances under the spans, and to the location of all river piers, should be obtained. Special permit drawings, 8 x 10½ in. (203 x 267 mm) in size, showing the pertinent data must be prepared. These requirements are given in the latest issue of *U.S. Coast Guard Bridge Permit Application Guide* of the appropriate district. Since the Coast Guard districts do not follow state boundaries, the address of the Coast Guard District having jurisdiction can be obtained by contacting Chief, Office of Navigation, U.S. Coast Guard (G-NBR), Washington, D.C. 20593, Phone - 202/755-7620.

In Canada such permit requirements can be obtained from Chief NWPA, Program Division, Transport Canada, Coast Guard, Ottawa, Ontario, Canada K1V 6T4.

2.7.2 Environmental laws and national policy

- a. The National Environmental Policy Act (NEPA) of 1969 (Public Law 91-190) requires that “all agencies of the Federal Government... include in every recommendation or report on major Federal actions significantly affecting the quality of the human environment, a detailed statement by the responsible official to outline:

“The environmental impact of the proposed action; any adverse environmental effects which cannot be avoided should the proposal be implemented; alternatives to the proposed action; the relationship between local short-term uses of man's environment and the maintenance and enhancement of long-term productivity; and any irreversible and irretrievable commitments of resources which would be involved in the proposed action should it be implemented.”

- b. Section 4(f) of the Department of Transportation Act (Public Law 89-670) declares that special effort should be made to preserve the natural beauty of the countryside and public park and recreational lands, wildlife and waterfowl refuges, and historic sites. The Secretary of Transportation...“shall not approve any program or project which requires the use of any publicly owned land from a public park, etc. unless (1) there is no feasible and prudent alternative to the use of such land, and (2) such program includes all possible planning to minimize harm to such park, recreational area, wildlife and waterfowl refuge, or historic site resulting from such use.”
- c. Historic preservation—The National Historic Preservation Act of 1966 and Executive Order 11593, Protection & Enhancement of Cultural Environment, require that Federal, or federally assisted projects must take into account the project's effect on any district, site, building, structure, or object that is included in the National Register of Historic Places and give the Advisory Council on Historic Preservation an opportunity to comment on the undertaking. Further, federal plans and programs should contribute to the preservation and enhancement of sites, structures, and objects of historical, architectural, or archeological significance.
- d. Clean Air Act—The impact of a bridge project on air quality must be assessed, and the project must be consistent with the state (air quality) implementation plan.

The Federal Highway Administration's policies and procedures for considering air quality impacts on highway projects are contained in FHWA *Program Manual*, Vol. 7, Chapter 7, Section 9 (Reference 2-10). This manual and any state or local standards may be used as a guide in determining the type of bridge projects for which air quality impacts are a reasonable concern.

- e. The Noise Control Act of 1972—This act establishes a national policy to promote an environment free from noise that jeopardizes health and welfare. For bridge projects where highway noise is a concern, FHWA *Program Manual*, Vol. 7, Chapter 7, Section 3 (Refer-

ence 2-10) and/or state or local standards may serve as a guide in evaluating and mitigating noise impacts.

- f. The Federal Water Pollution Control Act Amendments of 1972.—Section 401 requires that applicants for a federal permit provide a water quality certificate by the appropriate state or interstate agency. If there is no applicable effluent limitation and no standards, the state water quality certifying agency shall so certify. If the state or interstate agency fails or refuses to act on a request for certification within a reasonable length of time (normally deemed to be 3 months, but not to exceed 1 year) after receipt of request, the certification requirements shall be waived. No permit will be granted until certification has been obtained or waived, or if certification has been denied.

Section 404 assigns to the Corps of Engineers the responsibility for issuing permits for the discharge of dredged or fill material. However, the environmental documentation for a bridge project must contain an analysis of the impact of any fill associated with that project.

- g. Fish and Wildlife Coordination Act—Section 2 requires that, “whenever the water of any stream or other body of water are proposed or are authorized to be ... controlled or modified for any purpose whatever ... by any department or agency of the United States, or by any public or private agency under Federal Permit or license, such department or agency shall first consult with the United States Fish and Wildlife Service, Department of the Interior, and with the head of the agency exercising administration over the wildlife resources of the particular state where ... (the facility is to be constructed...)” The environmental documentation for the bridge project should include an analysis of probable impacts on fish and wildlife resources and an analysis of any mitigative measures considered, and adopted or rejected.
- h. The Endangered Species Act of 1973—This act generally provides a program for the conservation, protection, reclamation, and propagation of selected species of native fish, wildlife, and plants that are threatened with extinction. Section 7 of this act provides that federal agencies shall take “such actions necessary to insure that actions authorized, funded or carried out by them do not jeopardize the continued existence of such endangered species and threatened species or result in the destruction or modification of habitat of such species.” The list of endangered and threatened species, published by the Fish and Wildlife Service in the Federal Register, shall be consulted to determine if any species listed or their critical habitats may be affected by the proposed project. Section 7 of this act establishes a consultation procedure to avoid and mitigate impacts on listed species and their habitats.
- i. Water Bank Act—Section 2 of this Act declares that ... “It is in the public interest to preserve, restore and improve the wetlands of the Nation. Bridge projects must be planned, constructed, and operated to assure

protection, preservation, and enhancement of the nation's wetlands to the fullest extent practicable. Efforts should be made to consider alignments that would avoid or minimize impacts on wetlands, as well as design changes and construction and operation measures, to avoid or minimize impacts.

- j. Wild and Scenic Rivers Act—Section 7 of this act provides generally, that no license shall be issued for any water resources project where such project would have a direct and adverse effect on a river or the values for which such river was designated by this act. A bridge is considered to be included in the term “water resources project,” and a permit is a license.
- k. Prime and Unique Farmlands—Impacts of bridge projects on prime and unique farmlands, as designated by the State Soil Conservation service (U.S.D.A.), must be evaluated. Efforts should be made to assure that such farmlands are not irreversibly converted to other uses unless other national interests override the importance of preservation or otherwise outweigh the environmental benefits derived from their protection. Analysis of the impact of a bridge project on any such land SHALL be included in all environmental documents.
- l. Executive Order 11988, Floodplain Management—This Order sets forth directives to “avoid, to the extent possible, the long and short term impacts associated with the occupancy and modification of floodplains and to avoid direct or indirect support of floodplain development wherever there is a practicable alternative.” An analysis of a bridge project's effect on hydraulics should be included in the environmental documentation.
- m. Relocation assistance—The Uniform Relocation and Assistance and Real Property Acquisition Policies Act of 1970 applies to projects where federal funds are involved. If any federal funds are involved in a bridge project, the environmental documents shall show that relocated persons should be provided decent, safe, and sanitary housing; that such housing be available within a reasonable period of time before persons are displaced; that such housing is within the financial means of those displaced; and that it is reasonably convenient to public services and centers of employment.
- n. Executive Order 11990, Protection of Wetlands—Department of Transportation Policy is to avoid new construction in a wetland unless: (a) there is no practicable alternate to the construction, and (b) the proposed project includes all practicable measures to minimize harm to wetlands which may result from such construction.

Wetlands are defined as lands either permanently or intermittently covered or saturated with water. This includes, but is not limited to, swamps, marshes, bogs, sloughs, estuarine area, and shallow lakes and ponds with emergent vegetation. Areas covered with water for such a short time that there is no effect on moist-soil vegetation are not included in the definition, nor

are the permanent waters of streams, reservoirs, and deep lakes. The wetland ecosystem includes those areas which affect or are affected by the wetland area itself; e.g., adjacent uplands or regions up and down stream. An activity may affect the wetlands indirectly by impacting regions up or downstream from the wetland, or by disturbing the water table of the area in which the wetland lies.

2.7.3 Plans, specifications, and contracts—These engineering documents together should define the work expressly, clearly, thoroughly, and without possibility of ambiguous interpretation. The plans should show all dimensions of the finished structure, in necessary and sufficient details to permit realization of the full intent of the design and to facilitate the preparation of an accurate estimate of the quantities of materials and costs. The plans should also state which specification (e.g., AASHTO M77) was followed, the loading the bridge was designed to carry, any other special loading, the design strengths of materials (concrete, steel, bearings), the allowable and design footing pressures, the design method used (load factor or working stress), and the design flood. The construction specifications and contracts should also define construction methods, procedures, and tolerances to insure workmanship, quality control, and application of unit costs when stipulated under the contract. The Contractor's responsibilities should be clearly defined in detail, with everything expressly stated.

2.7.4 Construction inspection—The responsibilities of construction inspection for concrete bridges should always be clearly identified. Preferably, the owner should engage the designer of the bridge to inspect its construction, to review the contractor's procedures and falsework plans, which should be submitted prior to construction.

RECOMMENDED REFERENCES

The documents of the various standards-producing organizations referred to in this report are listed here with their serial designation, including year of adoption or revision. The documents listed were the latest effort at the time this report was written. Since some of these documents are revised frequently, generally in minor detail only, the user of this report should check directly with the sponsoring group if it is desired to refer to the latest revision.

American Association of State Highway and Transportation Officials

- HB-12 Standard Specifications for Highway Bridges, 13th Edition, 1983
- DS-2 Design Standards—Interstate System, 1967
- DSOF-3 Geometric Design Standards for Highways Other Than Freeways, 1969
- GD-2 A Policy on Geometric Design of Rural Highways, 1965
- GU-2 A Policy on Urban Highways and Arterial Streets, 1973
- HDG-7 Hydraulic Analysis for the Location and Design of Bridges, 1982

- GTB Guide for Selecting, Locating and Designing Traffic Barriers, 1977
- HM-14 Standard Specifications for Transportation Materials and Methods of Sampling and Testing

American Concrete Institute

- 345-82 Standard Practice for Concrete Highway Bridge Deck Construction
- 358R-80 State-of-the-Art Report on Concrete Guideways
- 504R-77 Guide to Joint Sealants for Concrete Structures
- SP-24 Models for Concrete Structures

American Railway Engineering Association Manual for Railway Engineering

- Chapter 8—Concrete Structures and Foundations
- Chapter 28—Clearances
- Chapter 29—Waterproofing

CITED REFERENCES

- 2-1. "Drainage of Highway Pavements," *Hydraulic Engineering Circular*, No. 12, Mar. 1969, U.S. DOT, FHWA.

- 2-2. "Location, Section and Maintenance of Highway Traffic Barriers," NCHRP Ref. #118—1971; TRB Washington, D.C.

- 2-3. "Handbook of Highway Safety Design and Operating Practices," U.S. DOT, FHWA, 1973.

- 2-4. "Hydraulic Design of Bridge with Risk Analysis," USDOT FHWA-T5-80-226, Mar. 1980.

- 2-5. "Bridge Aesthetics," a bibliography compiled by Martin P. Burke, Jr., P.E., published by Transportation Research Board.

- 2-6. FHWA Technical Advisory T 5140.12, Dec. 4, 1979, "Alternate Bridge Designs."

- 2-7. "Guidelines for Value Engineering," Task Force #19, AASHTO-AGC-ARTBA Joint Cooperative Committee.

- 2-8. "Value Engineering for Highways," Federal Highway Administration, U.S. Department of Transportation.

- 2-9. "Value Engineering in Construction," Department of the Army, Office of the Chief of Engineers, Sept. 1974.

- 2-10. "Federal-Aid Highway Program Manual," Federal Highway Administration, U.S. Department of Transportation.



CHAPTER 3—MATERIALS

3.1—Introduction

The ultimate realization and performance of concrete bridges depend upon well conceived and executed designs, skilled construction, and the use of reliable materials. Materials used in construction of concrete bridges are presented and/or referenced in this chapter. Also, recommended specifications for materials acceptance, sampling, and testing are presented and/or referenced.

3.2—Materials

3.2.1 Sources—Materials should be supplied from sources approved before shipment. The basis for approval should be the ability to produce materials of the quality and in the quantity required. These approved sources should be used as long as the materials continue to meet the requirements of the specifications. It is recommended that materials be in compliance with the standard specifications listed in the following sections.

The sources of the materials should be identified and contracts for their supply executed well in advance of the time when concreting is expected to begin. Sufficient lead time should be provided for the evaluating, sampling, and testing of all material if sources have not been previously approved by an appropriate agency, such as a State Department of Transportation.

3.2.2 Specifications and standard practices—Material specifications and tests for highway bridges should be in compliance with current AASHTO “Standard Specifications for Highway Bridges;” for railway bridges in compliance with current AREA “Manual for Railway Engineering,” Chapter 8. In addition, standard practices should be in accordance with ACI guidelines and ASTM material specifications.

3.2.3 Admixtures—Admixtures are materials used to modify the properties of concrete for a particular application. Generally, admixtures are employed to increase strength, improve workability and durability, and increase or decrease the time of setting. To insure the desired product, care should be exercised in selection, evaluation, and methods of addition. In evaluation, consideration should be given to the experience records of specific admixtures with concrete materials commonly used in the area. ACI 212.1R and 212.2R provide excellent resource information.

Air-entraining admixtures are used in bridge concrete to improve the durability of concrete in freeze-thaw cycling, particularly in the presence of deicing chemicals containing chlorides. Recommended air contents for various nominal coarse aggregate sizes are given in ACI 345. Specifications for air-entraining admixtures are given in ASTM C 260.

Water-reducing admixtures conforming to ASTM C 494, Type A or water-reducing and retarding admixtures conforming to ASTM C 494, Type D allow for a reduction in water-cement ratio and/or setting time for a given consistency of concrete. Concrete workability is maintained while strength is increased and concrete permeability reduced. Water-reducing admixtures in combination with air entrain-

ment will produce greater resistance to chlorides in a freeze-thaw environment. Additional economy will result where specifications permit reducing cement content.

High-range water-reducing admixtures, which reduce the quantity of mixing water by 12 percent or greater, are growing in use by manufacturers of precast concrete. Increased workability of the concrete mixture and accelerated compressive strength gain substantially reduce the time required to achieve stripping strengths. As in the case of conventional water-reducing admixtures, these admixtures can behave differently with different cements and temperatures. Two classes of high-range water-reducing admixtures are specified, denoting normal setting and retarding admixtures conforming to ASTM C 494, Types F and G, respectively.

Accelerating admixtures are used to increase high-early strength and decrease the setting time. Accelerators may be specified to facilitate early form removal or cold weather concreting.

Calcium chloride has been the most widely used accelerator since it is very effective and relatively economical. However, the use of calcium chloride in concrete promotes corrosion of metals in contact with it, due to the presence of chloride ions. Calcium chloride is not permitted where galvanized metal stay-in-place forms are used, or for use in prestressed concrete, or where dissimilar metals are embedded. It should not be used in any elements of concrete bridges/structures that may be exposed to additional chlorides. Calcium chloride should not be used as an admixture with lime-based slag cements, high-alumina cements, or with super-sulfated cements. Consideration should be given to obtaining the desired results without using an accelerating admixture, either by use of a water-reducer or high-early strength cement. AREA specifications do not permit the addition of calcium chloride. If used, calcium chloride should conform to ASTM D 98.

Calcium nitrite, although still under experimental evaluation by some states, can be added to reinforced concrete to inhibit corrosion and increase strength. Primary application has been with precast, prestressed concrete box and girder sections for bridges. The admixture should comply with ASTM C 494, Type C.

There are three classes of mineral admixture conforming to ASTM C 618: raw or calcined natural pozzolan (Class N), and two classes of fly ash (Class F and Class C). Class F has pozzolanic properties; Class C has some cementitious properties in addition to pozzolanic properties.

Mineral admixtures conforming to ASTM C 618 are used in concrete to reduce the heat of hydration in mass concrete, to improve the resistance of concrete to actions such as those caused by reactive aggregates, and to conserve cement by replacing a portion of the required cement except when high-early strength is required.

3.2.4 Aggregates—Aggregates for concrete consist of fine and coarse particles conforming to ASTM C 33. Both

AASHTO and AREA specifications require additional standard test methods. As a general guide, the maximum size of the aggregate should not be larger than one-fifth of the narrower dimension between sides of forms, one-third of the depth of slabs, or two-thirds (AREA specifies one-half) of the minimum clear spacing between reinforcing bars.

The sizes of the fine and coarse aggregates should be reasonably well graded from coarse to fine, and the maximum size of the coarse aggregate desired for specific structures should be specified. The sizes of coarse aggregates should conform to ASTM D 448. AREA specifications modify ASTM D 448 for the sizes of coarse aggregates.

Lightweight aggregates may be specified in concrete for structural elements. A prime consideration for bridge decks is a reduction of dead load by approximately 25 percent. Availability of lightweight aggregates at an economical price may be a concern in some areas. Abrasion and durability should be evaluated when lightweight aggregate is being considered for use in bridge decks or other exposed locations. Lightweight aggregates, if required or permitted by special provisions, should conform to ASTM C 330. AREA specifications modify ASTM C 330.

The basic physical and chemical characteristics of aggregate cannot be altered by processing, although the quantities of certain deleterious particles can be reduced. Preparation and handling affect such important aggregate properties as gradation, uniformity of moisture content, cleanliness, and in the case of crushed aggregate, particle shape, thereby having an important influence on concrete quality. Frozen aggregates or aggregates containing frozen lumps should be thawed before use. Aggregates should have a reasonably uniform moisture content when delivered to the mixer. Information covering selection and application of aggregates for concrete may be obtained from the report of ACI 221.

3.2.5 Cement—The type of cement should always be specified.

Cement should conform to one of the specifications for cement listed below:

- a. Portland cement types as designated in ASTM C 150
- b. Three shrinkage-compensating portland cements given in ASTM C 845 designated as Types K, M, and S. Recommended application is given in ACI 223.
- c. Portland blast-furnace slag cement or portland-pozzolan cement as given in ASTM C 595 for structural concrete.

The amount of cement used in the concrete should correspond to the specified job mixture.

3.2.6 Water—Water for washing aggregates and for use with cement in mortar or concrete should be clean and free from deleterious amounts of oil, salt, sugar, acid, alkali, organic materials, or other substances interfering with the required strength, density, impermeability, and durability of concrete and steel, or otherwise harmful to the finished product.

Nonpotable water should be tested and should meet the suggested limits of AASHTO T 26. Nonpotable water should not be used in concrete unless the following are satisfied: selection of concrete proportions shall be based on concrete mixes using water from the same source; mortar test

cubes made with nonpotable mixing water shall have 7-day and 28-day strengths equal to at least 90 percent of strengths of similar specimens made with potable water; strength test comparison shall be made on mortars, identical except for the mixing water, prepared and tested in accordance with ASTM C 109. Cement paste setting should conform to ASTM C 191, and concrete setting to ASTM C 403.

3.2.7 Selection of concrete proportions—Recommendations for selecting proportions for concrete are given in ACI 211.1. Provision is given for selecting and adjusting proportions for normal weight concrete by the estimated weight and/or the absolute volume methods.

Recommendations for lightweight concrete are given in ACI 211.2. Provision is made for proportioning and adjusting structural grade concrete containing lightweight aggregate.

Concrete ingredients and proportions should be selected to meet the minimum requirements stated in the specifications and contract documents. Field experience or laboratory trial mixes are the preferred methods for selecting concrete mixture proportions.

ACI 318 limits chloride ion content for corrosion protection, depending on member type. An initial evaluation can be obtained by testing individual concrete ingredients for total chloride ion content. Additional information is given in ACI 201.2R and ACI 222R on the effects of chlorides on the corrosion of reinforcing steel. When coated reinforcement steel is used, the preceding guidelines may be more restrictive than necessary.

3.2.8 Curing materials—Freshly cast concrete should be protected from premature drying and excessive heat or cold. To insure continued hydration at an optimum rate, the concrete should be kept saturated by wet, membrane, or steam curing for a given length of time. Preservation of the concrete moisture content may be accomplished by using burlap cloth, enclosure steam curing, liquid membrane-forming compounds, or waterproof sheet materials.

Burlap cloth should be made of jute or kenaf conforming to AASHTO M182. The cloth should remain wet for the entire specified curing time.

The liquid membrane-forming compounds should be suitable for spraying on horizontal and vertical surfaces and should conform to ASTM C 309. The compounds covered by this specification are available in different colors and are suitable for use as curing media for fresh concrete. They also provide additional curing of the concrete after removal of forms or after initial moist curing. The application of Type 2, white pigmented compound is generally preferred because it reduces the temperature rise in concrete exposed to the sun. However, at times its application may lead to an unsightly appearance since the compound does not wear off readily or evenly. Type 1-D with fugitive dye may be found more desirable for structural barriers or substructure work.

Waterproof sheet material should conform to ASTM C 71. This material is placed on the surfaces of concrete for minimizing moisture loss during the curing period, and in the case of the white reflective-type material, reducing temperature rise in concrete exposed to the sun. Early

application of this material may produce a smooth, glossy surface finish which is undesirable for a bridge deck.

The requirements of curing practice as prescribed by ACI 308 should be followed.

3.2.9 Joint materials—ACI 504R should be consulted for information on joint materials and construction.

3.2.9.1 Water stops—Water stops are used to prevent the infiltration of debris and water at construction, contraction, fixed and expansion joints. These are of metal, rubber, or plastics. Where the function of the joint is to provide movement, the water stops should be designed to permit such movement and be of a shape and thickness that will accommodate the force effects anticipated in the water stop. Spliced, welded, or soldered water stops should form continuous watertight joints.

Metal water stops may be made of copper or stainless steel strips. Copper water stops or flashings should conform to ASTM B 152 Copper No. 11000, electrolytic tough pitch type, light cold-rolled, soft annealed. Stainless steel water stops should conform to ASTM A 167, Type 316 L; No. 2 D Finish.

Rubber water stops may be molded or extruded. Their cross section should be uniform, free from porosity or other defects. The material for water stop may be compounded from natural rubber, synthetic rubber, or a blend of the two, together with other compatible materials which will produce a finished water stop conforming to the test requirements of ASTM D 412, D 572, D 746, D 747, D 792, and D 2240.

Plastic water stops should be fabricated by an extrusion process with a uniform cross section that is free from porosity or other defects. The material used for the water stop should be a homogeneous, elastomeric, plastic compound of basic polyvinyl chloride and other material which, after fabrication, should conform to the same ASTM test requirements as rubber water stops.

3.2.9.2 Joint fillers—Expansion joints may be filled with bituminous, cork, sponge rubber, or other approved expansion joint filler material. Preformed joints that are of bituminous type and are to be resilient should conform to ASTM D 1751. Preformed joints that are of bituminous type and are less than 1/2 in. thick, such as those used in parapet joints over piers of continuous structures, should conform to ASTM D 994.

Preformed joints that are nonbituminous should conform to ASTM D 1752, Type I, for sponge rubber material when resiliency is required and Type II for cork material when resiliency is not required.

Preformed joints that do not require the resiliency provided by rubber, and which are used for exposed concrete, may be made of polystyrene material conforming to AASHTO M 230. When the joint is not exposed, such as for a column hinged at a footing, the preformed material may consist of bituminous material conforming to ASTM D 994 or to cork material conforming to ASTM D 1752, Type II.

3.2.9.3 Joint sealants—Preformed expansion joints may be sealed with cold-application-type sealer, a hot-poured elastic type sealer, or an elastomeric-type joint seal. The cold-application joint sealer should conform to ASTM D 1850. The hot-poured elastic type should conform to ASTM

D 1190, and the elastomeric type should conform to AASHTO M 220.

3.2.9.4 Mechanically locked sealants—Numerous proprietary mechanically locked sealants in compression and/or tension are available. Prior to specifying sealants, test data and performance information should be obtained from the manufacturer, or an agency where testing or performance has been monitored and evaluated. Details for application and size selection are given in ACI 504R.

Mechanical locking of compression seals between armoring improves performance since direct compression alone cannot be relied on to keep the seal in place. For example, multi-unit modular joint systems, employing a number of transverse extruded steel sections having “locked in” elastomeric seals, are available. To insure joint sealing integrity, it is important to provide sufficient anchorage between the armor and concrete anchorage to withstand traffic impact. Good field inspection is necessary to insure proper installation.

Strip seals are widely used in concrete bridge decks for thermal movements up to 4 in. (100 mm). The preformed elastomeric seal element is mechanically locked between armored interfaces of extruded steel sections. During structure movements, a preformed central hinge enables the strip seal gland profile to fold between the steel extrusions. When properly sized and installed, watertightness is insured by wedge-action of the elastomeric lugs within the steel extrusions.

Steel used in the extrusions should conform to ASTM A 242, A 36, or A 588. The elastomer used in the seal or gland element should conform to the requirements of ASTM D 2628.

3.2.9.5 Steel joints—Due to the availability, effectiveness, and range of movement of mechanically locked glands or seals, open steel joints are becoming less attractive. Positive protection against joint leakage is required to prevent deterioration of bridge bearings and substructure units. Steel joints can be made watertight by specifying a neoprene trough. However, past experience indicates that regular maintenance is required to keep the trough free of debris.

Steel joints are either a sliding plate or finger type. The sliding is generally limited to thermal movements up to 4 in. (100 mm) and the steel finger type up to 8 in. (200 mm). The sliding plate and finger joints should be examined for possible warpage before installation by laying the plates together, loose, or on a flat surface. The steel should conform to ASTM A 36 or A 588. Neoprene, if used, should conform to ASTM D 412 and D 2240.

3.2.10 Bearings—The function of bridge bearings is to transfer loads from the superstructure to the substructure. Also, bearings should accommodate rotational movements of the superstructure elements. Bearing guidelines are in the process of being written by ACI Committee 554.

Expansion bearings should permit both longitudinal and rotational movements while transferring lateral loads such as wind. The coefficient of friction on mating surfaces should be as low as practical; high frictional forces may increase the cost of substructure units.

Fixed bearings should allow rotational movement while transferring both lateral and longitudinal loads to the

substructure units. Fixed bearings tie the superstructure to the substructure, thus preventing the bridge from potential down-grade translation.

Several types of bearings are available; some of the more common types are as follows:

3.2.10.1 Elastomeric bearings—Elastomeric bearings are either molded, single-unit laminated pads with integral layers of nonelastic shims or nonlaminated, molded, or extruded pads. The elastomer should be natural rubber or neoprene of Grade 55 + 5, durometer hardness material conforming to the tests of ASTM D 2240, D 412, D 573, D 395, D 1149, D 429, and D 746, Procedure B. Experience indicates that steel laminates are superior to other nonelastic laminates. Laminates should be rolled, mild steel sheets conforming to ASTM A 570, A 36, or A 611, Grade D. All components of laminated bearings should be molded together in an integral unit and all laminated edges should be covered by a minimum 1/8-in. (3-mm) thickness of elastomer.

3.2.10.2 PTFE slide bearings—Polytetrafluoroethylene (PTFE) slide bearings are self-lubricating and can be bonded to a rigid back-up material capable of resisting horizontal shear and bending stresses. Expansion bearings of PTFE are not recommended without providing an elastomer or rocker plate to accommodate rotation. Stainless steel or other equally corrosive-resistant material should be used for a smooth, low-friction mating surface. Stainless steel and unfilled PTFE made with virgin TFE resin should conform to ASTM A 240, Type 316 and D 1457, respectively.

3.2.10.3 Steel bearings—Steel bearings are either roller, rocker, sliding, or large built-up rocker types. Steel bearings are fabricated from materials conforming to ASTM A 36, A 572, or A 588. All structural steel bearing plates should be flat-rolled with smooth surfaces free of warp, having edges straight and vertical. If lubricated bronze plates are required, they should conform to ASTM B 22, Alloy 911 or B 100, Copper Alloy 510.

On painted structures, the upper 6 in. (150 mm) of anchor bolts, nuts, and washers should have a protective coating. If specified, galvanizing should conform to ASTM A 153 or B 633.

3.2.10.4 Pot bearings—Pot bearings consist of a circular steel piston and cylinder which confine an elastomeric pad. The elastomer is prevented from bulging by the pot and acts similarly to a fluid under pressure. The pot bearing serves as an economical alternate to built-up steel bearings for higher load applications. These bearings allow rotational movement either with or without lateral and longitudinal movement. Polyether urethane elastomer of pure virgin material conforming to ASTM D 2240, D 412, or D 395 is recommended for the rotational element. Longitudinal movement can be attained by incorporating a PTFE slide bearing. The steel cylinder and piston should preferably be machined from a single piece of steel conforming to ASTM A 588.

3.2.10.5 Shear inhibited disc bearings—Shear inhibited disc bearings consist of a load-bearing and rotational disc of polyether urethane enclosed between upper and lower steel bearing plates equipped with an internal shear restriction pin. Expansion is accommodated by using a recessed PTFE on

the upper half of the top bearing plate. The PTFE surface supports an upper steel plate having a stainless steel surface. The upper steel plate is fitted with guide bars to restrict lateral movement.

The rotational element should be molded from polyether urethane conforming to ASTM D 412, D 395, and D 2240. Steel and stainless steel should conform to ASTM A 36 and ASTM A 167, Type 316, respectively, unless otherwise specified in the contract documents. PTFE should be made from pure virgin unfilled teflon resin conforming to ASTM D 638 and D 792.

3.2.11 Metal reinforcement

3.2.11.1 Reinforcing bars—All reinforcing bars should be deformed except plain bars may be used for spirals or for dowels at expansion or contraction joints. Reinforcing bars should be the grades required by the contract documents, and should conform to one of the following specifications:

- a. ASTM A 615
- b. ASTM A 616, including Supplement SI
- c. ASTM A 617
- d. ASTM A 706

Billet-steel reinforcing bars conforming to ASTM A 615, Grade 60 [minimum yield strength 60 ksi (414 MPa)], are the most widely used type and grade. The current edition of ASTM A 615 covers bar sizes #3 through #11, #14, and #18. ASTM A 615M covers metric bar sizes #10 through #55.

When important or extensive welding is required, or when more bendability and controlled ductility are required as a seismic-resistant design, use of low-alloy reinforcing bars conforming to ASTM A 706 should be considered. Before specifying A 706 reinforcing bars, however, local availability should be investigated.

3.2.11.2 Coated reinforcing bars—When coated reinforcing bars are required as a corrosion-protection system, the engineer should specify whether the bars are to be zinc-coated (galvanized) or epoxy-coated. The reinforcing bars to be coated should conform to the specifications listed in Section 3.2.11.1.

- a. Zinc-coated (galvanized) reinforcing bars should conform to ASTM A 767. Supplementary requirement S1 should apply when fabrication of the galvanization includes cutting. Supplementary requirement S2 should apply when fabrication after galvanization includes bending.
- b. Epoxy-coated reinforcing bars should conform to ASTM A 775.
- c. Repair of damaged zinc coating, when required, should use a zinc-rich formulation conforming to ASTM A 767. Repair should be done in accordance with the material manufacturer's recommendations.
- d. Repair of damaged epoxy-coating, when required, should use a patching material conforming to ASTM A 775. Repair should be done in accordance with the material manufacturer's recommendations.

For zinc-coated reinforcing bars (galvanized) in accordance with ASTM A 767, the engineer should specify class of coating, whether galvanization is to be performed before or after fabrication, and if supplementary requirement S3

applies. If the bars are to be galvanized after fabrication, the engineer should indicate which bars require special finished bend diameters, such as the smaller bar sizes used for stirrups and ties. All other reinforcement and embedded steel items in contact with or in close proximity to galvanized reinforcing bars should be galvanized to prevent a possible reaction of dissimilar metals.

On projects using both coated and uncoated bars, the engineer should clearly indicate on the plans which bars are to be coated.

3.2.11.3 Bar mats—Bar mat reinforcement consists of two layers of deformed reinforcing bars assembled at right angles to each other, and clipped or welded at the bar intersections to form a grid. The reinforcing bars used in mats should conform to the specifications listed in Section 3.2.11.1. Whether clipped or welded mats are required depends on the size of the mat and the rigidity required for preserving the shape of the mat during handling. Clipping the bars should be adopted whenever possible, as welding decreases the fatigue strength of the bar steel because of the stress concentration effect of the weld. Bar mats should conform to ASTM A 184.

Clipped bar mats may be fabricated from zinc-coated (galvanized) reinforcing bars. Metal clips should be zinc-coated (galvanized). Nonmetallic clips may be used. Coating damage at the clipped intersections should be repaired in accordance with recommendations given in Section 3.2.11.2(c).

Clipped bar mats may be fabricated from epoxy-coated reinforcing bars. Metal clips should be epoxy-coated. Nonmetallic clips may be used. Coating damage at the clipped intersections should be repaired in accordance with the recommendations given in Section 3.2.11.2(d).

3.2.11.4 Wire—Wire should be plain or deformed wire as indicated in the contract documents. Spirals may be plain wire. For wire with a specified yield strength f_y exceeding 60,000 psi (414 MPa), f_y should be the stress corresponding to a strain of 0.35 percent.

Plain wire should conform to ASTM A 82.

Deformed wire should conform to ASTM A 496, size D4 and larger.

3.2.11.5 Welded wire fabric—Welded wire fabric is composed of cold-drawn steel wires, which may or may not be galvanized, and fabricated into sheets by the process of electrical-resistance welding. The finished material consists essentially of a series of longitudinal and transverse wires arranged substantially at right angles to each other, and welded together at all points of intersection. The use of this material for concrete reinforcement should preferably be at a location where the application of the loading is static or nearly so. If the application of the loading is highly cyclical, the fatigue strength of the wire fabric should be considered when determining the sizes of the wires. For fatigue considerations reference should be made to ACI 215R. The steel wires for the fabric may be either plain or deformed. The engineer should specify the size and wire spacing, and whether the fabric is to be plain or deformed. For wire with a specified yield strength f_y exceeding 60,000 psi (414 MPa), f_y should be the stress corresponding to a strain of 0.35 percent.

Welded plain wire fabric should conform to ASTM A 185, and welded intersections should not be spaced farther apart than 12 in. (300 mm) in the direction of the primary flexural reinforcement.

Welded deformed wire fabric should conform to ASTM A 497, and wire should not be smaller than size D4, and welding intersections should not be spaced farther apart than 16 in. (400 mm) in the direction of the primary flexural reinforcement.

Welded wire fabric made from a combination of either deformed or plain wires should conform to ASTM A 497 with the same exceptions previously listed for deformed fabric.

3.2.11.6 Prestressing tendons—Strands, wire, or high-strength bars are used for tendons in prestressed concrete.

a. Strands. Two grades of seven-wire, uncoated, stress-relieved steel strand are generally used in prestressed concrete construction. These are Grades 250 and 270, which have minimum ultimate strengths of 250,000 and 270,000 psi (172 and 186 MPa), respectively. The strands should conform to ASTM A 416. Supplement 1 of ASTM A 416 describes low-relaxation strand and relaxation testing. Recommended test procedures for stress relaxation are given in ASTM E 328.

b. Wire. Two types of uncoated stress-relieved round high-carbon steel wires commonly used in prestressed linear concrete construction are: Type BA wire for applications in which cold-end deformation is used for anchoring purposes (Button Anchorage) and Type WA wire for applications in which the ends are anchored by wedges and no cold-end deformation of the wire is involved (Wedge Anchorage).

Post-Tensioning systems commonly used are described in *Post-Tensioning Manual* by the Post-Tensioning Institute. Types WA and BA wire should conform to ASTM A 421. Supplement 1 of ASTM A 421 describes low-relaxation strand and relaxation testing. Recommended test procedures for stress relaxation are given in ASTM E 328.

c. Uncoated high-strength bars for prestressed concrete construction should conform to ASTM A 722. The specification covers two types of bars: Type I (plain) and Type II (deformed).

3.2.11.7 Structural steel, steel pipe, or tubing—Structural steel used with reinforcing bars in composite columns should conform to ASTM A 36, A 242, A 441, A 572, or A 588.

Steel pipe or tubing for composite columns should conform to ASTM A 53 (Grade B), A 500, or A 501.

3.2.12 Accessories

3.2.12.1 Bar supports—All reinforcement should be supported and fastened together to prevent displacement before and during casting of concrete. Bar supports may consist of concrete, metal, plastic, or other acceptable materials. Details and recommended practices for bar supports are given in ACI 315.

Standardized, factory-made steel wire bar supports are the most widely used type. Where the concrete surface will be exposed to the weather in the finished structure, the portions of bar supports near the surface should be protected against corrosion.

Bar supports for supporting zinc-coated (galvanized) or epoxy-coated reinforcing bars should be in accordance with the recommendations given in Section 13.11 of this document.

3.2.12.2 Side form spacers—Side form spacers, if needed, are placed against vertical forms to maintain prescribed clear concrete cover and position of the vertical reinforcing bars. The need for side form spacers is determined by the proportions of the form, the arrangement and placing of the reinforcing bars, the form material and forming systems used, and the exposure of the surface to weather and/or deleterious materials. In situations where spacers are needed, various devices can be used such as double-headed nails, form ties, slab or beam bolsters (wire bar supports), precast concrete blocks, proprietary all-plastic shapes, etc.

The engineer should specify the requirements for side form spacers including material, type, spacing, and location where required.

Section 1.9 of AREA Manual, Chapter 8, specifies that at all vertical formed surfaces that will be exposed to the weather in the finished structure, side form spacers spaced no further than 4 ft on center shall be provided. Spacers and all other accessories within 1/2 in. of the concrete surface shall be noncorrosive or protected against corrosion.

3.2.12.3 Tie wire—The tie wire used to fasten reinforcing bars should be black annealed steel wire, 16-gage (1.6-mm diameter) or heavier. Tie wire for fastening zinc-coated (galvanized) or epoxy-coated reinforcing bars should be in accordance with the recommendations given in Section 13.11 of this document.

3.2.12.4 Bar splicing material—When required or permitted, welded splices or mechanical connections may be used to splice reinforcing bars.

All welding of reinforcing bars should conform to “Structural Welding Code—Reinforcing Steel” AWS D1.4. Reinforcing bars to be welded should be indicated on the drawings and the welding procedure to be used should be specified. Except for ASTM A 706 bars, the engineer should specify if any more stringent requirements for chemical composition of reinforcing bars than those contained in the referenced ASTM specifications are desired, i.e., the chemical composition necessary to conform to the welding procedures specified in AWS D1.4.

Proprietary splice devices are available for making mechanical connections. Performance information and test data should be secured from manufacturers. Descriptions of the physical features and installation procedures for selected splice devices are given in ACI 439.3R.

3.2.12.5 Tensioning tendon components—Anchorages, couplers, and splices for post-tensioned reinforcement should develop the required nominal strength of the tendons, without exceeding the anticipated set. Anchorages for bonded tendons should develop at least 90 percent of the specified ultimate strength of the prestressing steel when tested in an unbonded condition, without exceeding the anticipated set. Couplers and splices should be placed in areas approved by the engineer and enclosed in housings long enough to permit the necessary movements. They should not be used at points of sharp curvature and should be staggered.

Performance specifications for single unbonded strand post-tensioning tendons are provided in the *Post-Tensioning Manual* of the Post-Tensioning Institute. Unbonded tendon anchorages should be subjected to the following additional requirements as recommended in ACI 423.3R.

- a. *Static tests*—When an assembly consisting of the tendon and fittings is statically loaded, it should meet the requirements set forth in the ASTM tendon material specifications for yield strength, ultimate strength, and minimum elongation. If minimum elongation at rupture is not stated by ASTM specifications, the elongation of the assembly should not be less than 3 percent measured on not less than a 10-ft (3-m) gage length.
- b. *Cyclic tests*—The test assembly should withstand, without failure, 500,000 complete cycles ranging between 60 and 66 percent of the specified ultimate strength.
- c. When used in structures subjected to earthquake loadings, the test assembly should withstand, without failure, a minimum of 50 complete cycles of loading corresponding to the following percentages of the minimums specified ultimate strength in ksi (MPa)

$$60 + 2000/l_n + 100$$

$$(60 + 610/l_n + 30.5)$$

where l_n is the length of the tendon in feet (meters).

- d) Kinking and possible notching effects by the anchorage should be avoided, particularly where tendon assemblies will be subjected to repetitive or seismic loadings.

3.2.13 Appurtenances

3.2.13.1 General—Materials in contact with and partially embedded in concrete should be so constituted as to not be injurious to the concrete.

3.2.13.2 Forms—Metal forms that are to remain in place should be zinc-coated galvanized, both for appearance and durability. Material for metal forms should conform to ASTM A 446 Coating Designation G165. Any exposed form metal where the zinc coating has been damaged should be thoroughly cleaned and wire brushed, then painted with two coats of zinc oxide-zinc dust primer, Federal Specification TT-P-641d, Type II, no color added.

Details of formwork, design criteria and descriptions of common types are shown in ACI SP-4.

3.2.13.3 Form coatings—Oil or other types of coating used on forms to prevent sticking of the concrete should not cause softening or permanent staining of the concrete surface, nor should it interfere with any curing process which might be used after form removal. Surfaces of forms made with lumber containing excessive tannin or other organic substance sufficient to cause softening of the concrete surface should be treated with whitewash or limewater prior to applying the form oil coating. Shellac, lacquers, and compounded petroleum oils are commercially available for form coating. The manufacturer of the coating should certify that the product will not be deleterious to concrete.

3.2.13.4 Galvanized materials—When used, nuts, washers for anchoring, and posts, luminaires, pedes-

trian chain link fences, ladders, stairways, joint dams, bearings, as well as junction boxes, conduits and fittings, and exposed steel inserts are galvanized, the protective zinc covering should conform to ASTM A 153. When door frames and doors provided for access to cells of box girders, operator machinery, pump rooms, and floor drains fabricated of structural steel are galvanized, the protective zinc covering should conform to ASTM A 123.

3.2.13.5 Cast iron and stainless steel—The material for cast iron hardware such as scuppers and drains, embedded in concrete, should preferably conform to ASTM A 48. When steel fasteners are embedded in concrete and are subjected to alternate wetting and drying, the material should preferably consist of stainless steel conforming to ASTM A 276, Type 316.

3.2.14 Storage of materials

3.2.14.1 Cement—The handling and storage of cement should be such as to prevent its deterioration or the intrusion of foreign matter and moisture. The recommendations given in ACI 304 should be followed.

3.2.14.2 Aggregates—Aggregates should be handled and stored in such a manner as to minimize segregation, degradation, contamination, or mixing of different kinds and sizes. When specified, the coarse aggregate should be separated into two or more sizes to secure greater uniformity of the concrete mixture. Different sizes of aggregate should be stored in separate stock piles sufficiently removed from each other to prevent the material at the edges of the piles from becoming intermixed.

3.2.14.3 Metal reinforcement—Reinforcement should be stored above the surface of the ground and should be protected from mechanical injury and surface deterioration caused by exposure to conditions producing rust.

Prestressing tendons should be protected at all times against physical damage and corrosion, from manufacture to either grouting or encasing in concrete. Prestressing tendons should be packaged in containers or shipping forms for the protection of the steel against physical damage and corrosion during shipping and storage. A corrosion inhibitor which prevents rust or other results of corrosion should be placed in the package or form, or the tendons should be precoated with water-soluble oil. The corrosion inhibitor should have no deleterious effect on the steel or concrete or bond strength of steel to concrete. Care should be taken in the storage of prestressing tendons to prevent galvanic action.

3.3—Properties

Concrete is not a homogeneous material. Its properties cannot be predicted with accuracy. They vary not only with the ingredients that make up concrete, but with the method of mixing, placing, and even the loading. The design engineer has only partial control of these variables, so it is necessary to understand them, the probable range of their variation, and their effect on the bridge structure. Only with a thorough understanding can the engineer design and detail a structure that functions properly.

3.3.1 Compressive strength—The design engineer specifies the strength of concrete needed to insure the structural

adequacy. In the past it was always considered that the stronger the concrete the better. Today's designer should realize that some of the other properties that are directly related to the strength of the concrete may not make it desirable to allow concrete significantly stronger than specified.

3.3.2 Tensile strength—In determining the strength of a structure, the tensile strength of concrete is not directly considered. However, it can be a significant factor in deflection computations.

3.3.3 Modulus of elasticity and Poisson's ratio—The modulus of elasticity for most bridge structures, using normal weight concrete [w_c between 90 and 150 pcf (1400 and 2400 kg/m³)], can be assumed to be that given in ACI 318-83

$$E_c = w_c^{1.5} 33 f'_c \text{ psi} \quad (E_c = w_c^{1.5} 0.043 f'_c \text{ MPa})$$

However, in ACI 363R-84, attention is called to the significant difference between actual tests made on many high-strength concretes and the ACI 318 formula.

If the structure being designed is of a significant size or if deflections are of significant importance, the designer should consider requiring tests to be made of the actual design mixture that would be used in the structure. Using the results of these tests in predicting deflections will be cost-effective, especially considering the time and effort necessary to correct the effects of improper camber and incorrectly predicted deflections.

Poisson's ratio may be assumed to be 0.2.

3.3.4 Creep—The inelastic strain of concrete under compressive stresses is generally a beneficial property. In this manner an indeterminate structure is able to adjust the dead load stresses to more efficiently utilize the inherent strength of the structure. However, when deflections due to creep become significant, it should be considered in the design. Creep decreases as the concrete ages, so one way to partially control creep is to prevent early loading of the structure or member. Creep also decreases with an increase in compressive reinforcement.

Creep can be as significant a property as the modulus of elasticity and should always be considered by the designer. Reference information for creep and shrinkage data is given in ACI 209R, Chapter 5.

3.3.5 Shrinkage—During the drying process concrete shrinks. The earlier the age that the concrete is dried, the greater the shrinkage. Since concrete is rarely exposed uniformly, the shrinkage of concrete is not uniform and internal shrinkage stresses are induced. Also, concrete sections are not restrained equally. This is particularly true at construction joints where fresh concrete is restrained by the previously cast concrete. The location of construction joints, the rate of concrete placement, and the concrete placing sequence can have a significant influence on the size and number of shrinkage-related cracks. The designer should be aware of the effects of shrinkage and creep and detail the structure to minimize the adverse effects.

The shrinkage coefficient for normal weight concrete should be established considering the local environmental conditions and the construction procedures. However, it

should not be taken as less than an equivalent strain of 0.0002. Shrinkage coefficients for lightweight concrete should be determined for the type of aggregate used.

3.3.6 Thermal coefficient—Concrete and steel have approximately the same thermal coefficient of expansion so that within the normal ranges the two materials act together. If temperature changes were unrestrained and uniform, there would be no need for concern. However, most concrete bridges have some degree of restraint, whether built-in or resulting from poor maintenance, deterioration of bearings and expansion details, or horizontal movements of supports. In addition, concrete is not a good thermal conductor. This lack of conductivity results in significant temperature differentials, particularly between the upper and lower surfaces of the bridge. Thus stresses due to temperature effects are induced that can be of great significance.

Unlike creep and shrinkage, temperature effects are both shortening and lengthening; but because temperature shortening is additive to creep and shrinkage, the shortening effect is generally more critical. Unlike shrinkage and dead load stresses, the temperature effect is not a sustained load, and thus cannot be alleviated by creep. A method of calculating temperature differentials is given in Section 5.4.

The thermal coefficient for normal weight concrete may be taken as 0.000006 F (0.000011 C). Thermal coefficients for lightweight concrete should be determined for the type of aggregate used.

3.3.7 State-of-the-art—For structures of greater than usual size, significance, or degree of complexity, the designer should consult the latest publications of ACI Committees: 209 Creep and Shrinkage, 363 High Strength Concrete, and 213 Lightweight Aggregates and Concrete. Currently these include SP-9, SP-27, SP-29, SP-76, ACI 209R, ACI 213R, and ACI 363R.

3.3.8 Reinforcement properties—The modulus of elasticity of deformed steel reinforcing bars or welded wire fabric may be taken as 29,000,000 psi (200,000 MPa). The modulus of elasticity for prestressing bars, strands, or wires should be determined by tests if not supplied by the manufacturer. When that information is not available, the following values may be used:

Bars: 28,000,000 psi (193,000 MPa)

Strands: 27,000,000 psi (186,000 MPa)

Wires: 29,000,000 psi (200,000 MPa)

3.4—Standard specifications and practices

The standard specifications for materials and practices referred to either directly or indirectly in this chapter are listed here. These specifications are constantly reviewed and revised by members of the respective technical committees. It is recommended that documents with the most recent dates of issue be reviewed in conjunction with previous standard specifications; the latest revisions are not always in conformance with the requirements of the various agencies and owners. A list of ACI guidelines and standard practices is given for reference. An ASTM-AASHTO specification cross-reference is given at the end of this chapter for ease of

comparing the two specifications. In many cases, however, there are significant differences between them.

3.4.1 ACI guidelines and standard practices

116R	Cement and Concrete Terminology
201.2R	Guide to Durable Concrete
209R	Predictions of Creep, Shrinkage and Temperature Effects in Concrete Structures
211.1	Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete
212.1R	Admixtures for Concrete
212.2R	Guide for Use of Admixtures in Concrete
213R	Guide for Structural Lightweight Aggregate Concrete
215R	Considerations for Design of Concrete Structures Subject to Fatigue Loading
221R	Guide for the Use of Normal Weight Aggregates in Concrete
222R	Corrosion of Metals in Concrete
223	Standard Practice for the Use of Shrinkage-Compensating Concrete
304	Recommended Practice for Measuring, Mixing, Transporting, and Placing Concrete
308	Standard Practice for Curing Concrete
315	Details and Detailing of Concrete Reinforcement
318	Building Code Requirements for Structural Concrete
345	Standard Practice for Concrete Highway Bridge Deck Construction
363R	Report on High-Strength Concrete
423.3R	Recommendations for Concrete Members Prestressed with Unbonded Tendons
439.3R	Mechanical Connections of Reinforcing Bars
504R	Guide to Joint Sealants for Concrete Structures
554	Committee Correspondence on Bearing Systems
SP-4	Formwork for Concrete
SP-9(OP)	Symposium on Creep of Concrete
SP-27(OP)	Designing for Effects of Creep, Shrinkage, and Temperature in Concrete Structures
SP-29(OP)	Lightweight Concrete
SP-76	Designing for Creep and Shrinkage in Concrete Structures

3.4.2 AREA Manual for Railway Engineering

Chapter 8—Concrete Structures and Foundations, 1985.

3.4.3 ASTM standards

A 36	Specification for Structural Steel
A 48	Specification for Gray Iron Castings
A 53	Specification for Pipe, Steel, Black and Hot-dipped, Zinc-Coated Welded and Seamless
A 82	Specification for Cold-Drawn Steel Wire for Concrete Reinforcement
A 123	Specification for Zinc (Hot-Galvanized) Coatings on Products Fabricated from Rolled, Pressed, and Forged Steel Shapes, Plates, Bars and Strip

A 153	Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware	B 100	Specification for Rolled Copper-Alloy Bearing and Expansion Plates and Sheets for Bridge and Other Structural Uses
A 167	Specification for Stainless and Heat-Resisting Chromium-Nickel Steel Plate, Sheet, and Strip	B 152	Specification for Copper, Sheet, Strip, Plate, and Rolled Bar
A 184	Specification for Fabricated Deformed Steel Bar Mats for Concrete Reinforcement	B 633	Specification for Electrodeposited Coatings of Zinc on Iron and Steel
A 185	Specification for Welded Steel Wire Fabric for Concrete Reinforcement	C 33	Specification for Concrete Aggregates
A 242	Specification for High-Strength Low-Alloy Structural Steel	C 109	Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or 50-mm Cube Specimens)
A 276	Specification for Stainless and Heat-Resisting Steel Bars and Shapes	C 150	Specification for Portland Cement
A 416	Specification for Uncoated Seven-Wire Stress-Relieved Steel Strand for Prestressed Concrete	C 171	Specification for Sheet Materials for Curing Concrete
A 421	Specification for Uncoated Stress-Relieved Wire for Prestressed Concrete	C 191	Test Method for Time of Setting of Hydraulic Cement by Vicat Needle
A 441	Specification for High-Strength Low-Alloy Structural Manganese Vanadium Steel	C 260	Specification for Air-Entraining Admixtures for Concrete
A 446	Specification for Steel Sheet, Zinc-Coated (Galvanized) by the Hot-Dip Process, Structural (Physical) Quality	C 309	Specification for Liquid Membrane-Forming Compounds for Curing Concrete
A 496	Specification for Steel Wire, Deformed, for Concrete Reinforcement	C 330	Specification for Lightweight Aggregates for Structural Concrete
A 497	Specification for Welded Deformed Steel Wire Fabric for Concrete Reinforcement	C 403	Test Method for Time of Setting of Concrete Mixtures by Penetration Resistance
A 500	Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes	C 494	Specification for Chemical Admixtures for Concrete
A 501	Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing	C 595	Specification for Blended Hydraulic Cements
A 570	Specification for Hot-Rolled Carbon Steel Sheet and Strip, Structural Quality	C 618	Specification for Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete
A 572	Specification for High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality	C 845	Specification for Expansive Hydraulic Cement
A 588	Specification for High-Strength Low-Alloy Structural Steel with 50 ksi (345 MPa) Minimum Yield Point to 4 in. Thick	D 98	Specification for Calcium Chloride
A 615	Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement	D 395	Test Methods for Rubber Property—Compression Set
A 615M	Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement (Metric)	D 412	Test Methods for Rubber Properties in Tension
A 616	Specification for Rail-Steel Deformed and Plain Bars for Concrete Reinforcement	D 413	Test Methods for Rubber Property—Adhesion to Flexible Substrate
A 617	Specification for Axle-Steel Deformed and Plain Bars for Concrete Reinforcement	D 429	Test Methods for Rubber Property—Adhesion to Rigid Substrates
A 706	Specification for Low-Alloy Deformed Bars for Concrete Reinforcement	D 448	Specification for Standard Sizes of Coarse Aggregate for Highway Construction
A 709	Specification for Structural Steel for Bridges	D 496	Specification for Chip Soap
A 722	Specification for Uncoated High-Strength Steel Bar for Prestressing Concrete	D 572	Test Method for Rubber Deterioration by Heat and Oxygen
A 767	Specification for Zinc-Coated (Galvanized) Bars for Concrete Reinforcement	D 573	Test Method for Rubber Deterioration in an Air Oven
A 775	Specification for Epoxy-Coated Reinforcing Steel Bars	D 746	Test Method for Brittleness Temperature of Plastics and Elastomers by Impact
B 22	Specification for Bronze Castings for Bridges and Turntables	D 747	Test Method for Stiffness of Plastics by Means of a Cantilever Beam
		D 792	Test Methods for Specific Gravity and Density of Plastics by Displacement
		D 994	Specification for Preformed Expansion Joint Filler for Concrete (Bituminous Type)
		D 1149	Test Method for Rubber Deterioration—Surface Ozone Cracking in a Chamber (Flat Specimens)

D 1190	Specification for Concrete Joint Sealer, Hot-Poured Elastic Type	M 173	Concrete Joint Sealer, Hot-Poured Elastic Type
D 1457	Specification for PTFE Molding and Extrusion Materials	M 182	Burlap Cloth Made from Jute or Kenaf
D 1751	Specification for Preformed Expansion Joint Fillers for Concrete Paving and Structural Construction (Nonextruding and Resilient Bituminous Types)	M 183	Structural Steel
		M 194	Chemical Admixtures for Concrete
		M 195	Lightweight Aggregates for Structural Concrete
D 1752	Specification for Preformed Sponge Rubber and Cork Expansion Joint Fillers for Concrete Paving and Structural Construction	M 203	Uncoated Seven-Wire Stress-Relieved Strand for Prestressed Concrete
		M 204	Uncoated Stress-Relieved Wire for Prestressed Concrete
D 1850	Specification for Concrete Joint Sealer, Cold-Application Type	M 213	Preformed Expansion Joint Fillers for Concrete Paving and Structural Construction
D 2240	Test Method for Rubber Property—Durometer Hardness	M 220	Preformed Elastomeric Compression Joint Seals for Concrete
D 2628	Specification for Preformed Polychloroprene Elastomeric Joint Seals for Concrete Pavements	M 221	Welded Deformed Steel Wire Fabric for Concrete Reinforcement
E 328	Recommended Practice for Stress-Relaxation Tests for Materials and Structures	M 222	High-Strength Low-Alloy Structural Steel with 50,000 psi Minimum-Yield Point to 4-in. Thick

3.4.4 AASHTO materials specifications

M 6	Fine Aggregate for Portland Cement Concrete
M 31	Deformed and Plain Billet-Steel Bars for Concrete Reinforcement
M 32	Cold-Drawn Steel Wire for Concrete
M 33	Preformed Expansion Joint Filler for Concrete (Bituminous Type)
M 42	Rail-Steel Deformed and Plain Bars for Concrete Reinforcement
M 43	Standard Sizes of Coarse Aggregate for Highway Construction
M 45	Aggregate for Masonry Mortar
M 53	Axle-Steel Deformed and Plain Bars for Concrete Reinforcement
M 54	Fabricated Steel Bar or Rod Mats for Concrete Reinforcement
M 55	Welded Steel Wire Fabric for Concrete Reinforcement
M 80	Coarse Aggregate for Portland Cement Concrete
M 85	Portland Cement
M 105	Gray Iron Castings
M 107	Bronze Castings for Bridges and Turntables
M 108	Rolled Copper-Alloy Bearing and Expansion Plates and Sheets for Bridge and Other Industrial Uses
M 111	Zinc (Hot-Galvanized) Coatings on Products Fabricated from Rolled, Pressed, and Forged Steel Shapes, Plates, Bars, and Strip
M 138	Copper Sheet, Strip, Plate, and Rolled Bar
M 144	Calcium Chloride
M 148	Liquid Membrane-Forming Compounds for Curing Concrete
M 153	Preformed Sponge Rubber and Cork Expansion Joint Fillers for Concrete Paving and Structural Construction
M 154	Air-Entraining Admixtures for Concrete
M 171	Sheet Materials for Curing Concrete

M 223	High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality
M 225	Deformed Steel Wire for Concrete Reinforcement
M 230	Extruded Insulation Board (Polystyrene)
M 232	Zinc Coating (Hot-Dip) on Iron and Steel Hardware
M 240	Blended Hydraulic Cements
M 251	Laminated Elastomeric Bridge Bearings
M 275	Uncoated High-Strength Steel Bar for Prestressing Concrete
T 26	Quality of Water to be Used in Concrete

3.4.5 ASTM-AASHTO specification cross-reference

ASTM specification	AASHTO specification
A 36	M 183
A 48	M 105
A 82	M 32
A 123	M 111
A 153	M 232
A 184	M 54
A 185	M 55
A 416	M 203
A 421	M 204
A 496	M 225
A 497	M 221
A 572	M 223
A 588	M 222
A 615	M 31
A 615M	M 31M
A 616	M 42
A 617	M 53
A 709	M 270
A 722	M 275
B 22	M 107
B 100	M 108
B 15	M 138
C 150	M 148



C 171	M 171
C 260	M 154
C 309	M 148
C 33	M 194
C 494	M 195
C 595	M 240
C 618	M 295
D 98	M 144
D 448	M 43
D 994	M 33
D 1190	M 173
D 1751	M 213
D 1752	M 153
D 2628	M 220

RECOMMENDED REFERENCES

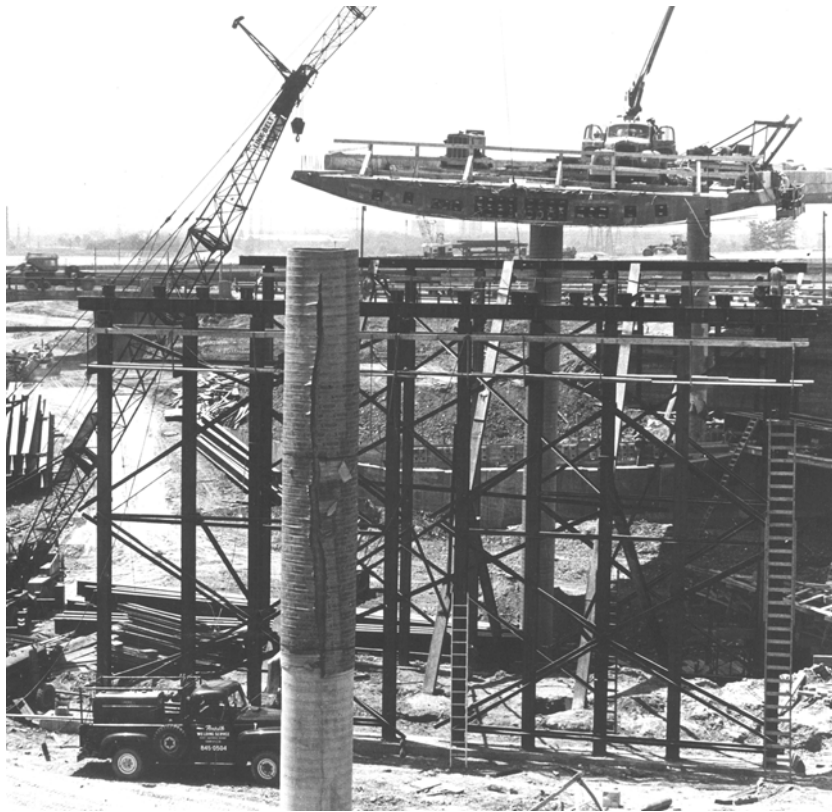
The documents of the various standards-producing organizations referred to in this report are listed below with their serial designation, including year of adoption or revision. The documents listed were the latest effort at the time this report was written. Since some of these documents are revised frequently, generally in minor detail only, the user of this report should check directly with the sponsoring group if it is desired to refer to the latest revision.

American Association of State Highway and Transportation Officials

Standard Specifications for Highway Bridges, Thirteenth Edition, 1983.

American Welding Society

D1.4-79 Structural Welding Code Reinforcing Steel



Complex construction site (photo courtesy of Ministry of Transportation and Communications, Ontario)

CHAPTER 4—CONSTRUCTION CONSIDERATIONS

4.1—Introduction

Every project progresses through various stages of development, starting with the recognition of the need and ending with a completed structure. The major steps in the evolution of a structure are:

- a. Preliminary engineering
- b. Design
- c. Construction

It is obvious that each phase of development affects the activities within the other phases. This chapter explores the influence of construction methods and procedures upon the design process.

4.1.1 Definition—Construction considerations may be defined as those details, procedures, and construction sequences that should be incorporated into design to cope with construction restrictions and limitations while insuring the constructibility of a structure. For quality assurance systems ACI 121R provides guidelines in establishing a project program for describing an organization's policies, practices, and procedures to comply with the contract documents.

4.1.2 Examples⁴⁻¹—Failure to consider construction restrictions during design generally results in additional expense and loss of time in rectifying the situation during construction, or unexpected maintenance expenses after the bridge is in service.

4.1.2.1 Section size—Some obvious construction considerations are involved in the design of a concrete

section. The section should be large enough not only to carry the imposed loads, but to physically accommodate the required number of reinforcing bars or prestressing tendons, while providing proper clearances and concrete cover. Thus, the designer should consider the maximum size of aggregate and the adequate size of a vibrator to be used by the contractor for proper placement and consolidation of concrete.

If a poor choice of section size is made, a more exacting concrete placement, a revision of maximum aggregate size, or a design of the section may become necessary. A change order may be required with consequent delay of the project and probable cost increase.

4.1.2.2 Camber—More complex construction considerations are involved in the determination of camber in structures constructed in stages (Fig. 4.1.2.2). Several factors have to be assessed quantitatively:

- a. Variation of modulus of elasticity of concrete with time
- b. Variation in creep rates depending on age of concrete at the time of loading
- c. Variation in creep rates for downward and upward deflections.
- d. Changing loading conditions
- e. Effects of temperature variations

If improper camber calculations are made, the appearance and riding quality of the deck will be affected. The condition will become obvious some time after the completion of the

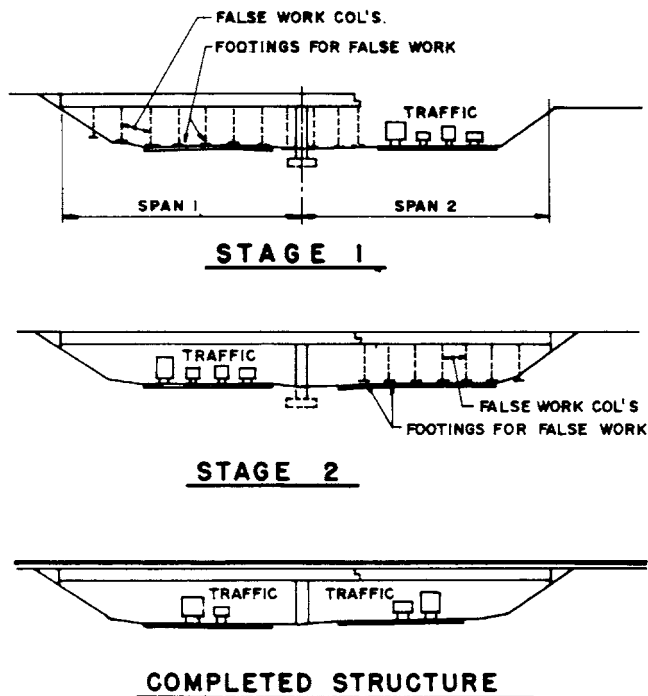


Fig. 4.1.2.2—Example of staged construction.

structure. The defect, although annoying, will seldom be serious. Since corrective measures are expensive, they will rarely be taken.

4.1.2.3 Construction sequence—The construction sequence is a very important consideration for large, continuous, multispan structures. A typical example is a composite precast, prestressed I-girder bridge with cast-in-place concrete deck. If the dimensions of the structure make it impossible to place the whole deck in one working day, it becomes necessary to provide a concrete placement diagram, specifying sequence and size of individual deck placement sections. The factors to consider are:

- Maximum size of a reasonable deck placement section
- Stresses at critical points during all stages
- Time interval between various sections
- Strength of concrete to be attained in one section before another one is allowed
- Effect of staged construction on deflections

If construction sequencing is neglected, the results can be serious. During placement of some of the deck sections, the stresses at critical points in the partially completed girder may exceed the design stresses, which are based on the assumption of continuity and composite action of the girder and the slab.

4.2—Restrictions

The construction phase must be considered during design to insure that the construction can be economically done in accordance with the design assumptions and to comply with construction restrictions imposed on the project by existing conditions or approving agencies.

Construction considerations originating in design assumptions usually deal with:

- Construction tolerances
- Stresses and deflections induced by construction sequences
- Effects of construction joint locations
- Measures to insure economy of construction

The construction considerations, having their source in restrictions imposed on the project, are numerous and may deal with any facet of the structure. The following tabulation gives examples of typical construction restrictions and their sources:

Owner—Project schedule (accelerated schedule will influence selection of the type of structure and construction sequence).

Approving agencies—Construction clearances (these are sometimes more restrictive than clearances for the completed structure, thus influencing layout and type of structure selection).

Timing of specific construction activities—Encroachments on the flood plain, construction in forests during fire season, pile driving or other noisy activities next to hospitals and schools.

Access restrictions—Parks, city streets.

Maintenance of traffic—More and more structures are being built as replacements or widenings.

Site characteristics—Accessibility (narrow, twisting roads preclude use of long, prefabricated girders).

Adjacent structures—There may be weight and size limitation.

Climate—Short construction season will influence type of structure selection.

Materials—Quality of local aggregates may determine maximum strength of concrete that can be readily produced (lightweight aggregates, freeze-and-thaw-resistant aggregates are not available at many locations).

Project needs—Traffic requirements may require staged construction (traffic detours).

Avoidance of possible claims—Blasting, dewatering, pile driving (in many cases proper design can minimize or even eliminate these problems).

4.3—Goals

The designer should strive to achieve four primary goals in the completed structure: sufficient capacity, dependable durability, economy of construction, and pleasing appearance. In general, sufficient capacity in structures is achieved by adhering to appropriate codes, although it should be kept in mind that most codes are based on an acceptable minimum capacity while quite often a greater capacity may be warranted or desirable. The achievement of any of the goals requires engineering judgment.

The goals are interrelated; effort expended to better one goal invariably changes the others, but not necessarily for the worse. The goals are also influenced by availability of materials and by construction methods.

The ideal of constructing a bridge with an excess of capacity that is maintenance free and has a very pleasing appearance, at much less than the budget figure, is seldom attainable. Various factors have to be considered to determine the effect of each construction consideration on each goal. Trade-offs must be weighed to achieve the optimum result.

The importance of each of the goals varies with the political and social environment, as well as the geographical location of the project. Adequate capacity must always be insured. The appearance of a bridge is usually a more sensitive issue in an urban area location than in an industrial or a rural area (although obviously each case has to be considered and decided on its own merits).

When cost of construction is of prime importance, construction methods tend to control design. The designer should consider several methods to select the most economical way to build a particular project, recognizing in each method the applicable restrictions mentioned in the previous section. Once the most economical method and material has been determined, the designer decides how to use them to achieve acceptable appearance and adequate capacity and durability.

New construction methods and material strengths often create situations in which construction loads may govern the required strength of part or all of a structure. The designer should always keep construction loadings in mind. A close working relationship with qualified contractors should be developed to insure that allowable stresses are not exceeded during construction. Contractors are becoming increasingly involved with engineering requirements. The most efficient use of labor and materials leads to more engineering-intensive designs that require closer tolerances and higher stresses. The level of sophistication in design should take into account a reasonable level of the quality of construction that can be achieved.

In summary, the design engineer should recognize the applicable construction considerations during the design phase; the construction engineer should recognize the implications of the design criteria and be able to construct the project within these limitations. The result will be an economical product that is adequate in capacity, pleasing in appearance, and durable in service.

4.4—Planning

A conventionally scheduled design project, as opposed to fast-track projects, is characterized by a sequential progression in which each activity depends on work performed in a previous one. Because of this dependence, each phase should be completed before the next is begun. The design process should include the following activities:

- a. Data collection
 1. Traffic during construction
 2. Falsework
 3. Stage construction requirements
 4. Construction restrictions
 5. Construction loads
 6. Allowable construction stresses
- b. Layout
- c. Foundations
- d. Structure type selection
- e. Details

As this list demonstrates, most construction considerations are based on data collected during the initial phase of design. Any construction requirement or restriction overlooked, during this period, may be a costly item to consider at a later stage.

Costs of revisions escalate, particularly after various binding decisions on layout or type of structure have been made.

Thus, it is essential to deal with all construction considerations affected by external constraints at the earliest stages of project activities. On the other hand, construction considerations originating in design assumptions may be handled when the affected detail is being designed.

4.5—Site characteristics

In the past “forced solutions,” using standard right angle structures requiring the realignment of streams or secondary roads, were often adopted under the guise of economy. Today it is axiomatic that the structure should fit the site. Some of the elements dictating the choice of structure include alignment, length, spans, depth, and foundations. Lately, with greater emphasis on esthetics, the requirement of visual harmony with the site in terms of shape, color, and texture has been added to this list. In addition, some site characteristics affect the design indirectly by placing various restrictions on construction.

4.5.1 Site accessibility—Many bridge construction sites are remote, and accessible only by narrow, winding roads. Such conditions generally favor structures that require minimum amounts of materials and field labor. When poor roads preclude transportation of long prefabricated girders, girders may be shipped in segments and assembled at the site.

4.5.2 Climate—The various ways climate influences construction should be considered in design.

4.5.2.1 General—In areas of severe climate, the construction season may be short, favoring structures that allow a great deal of shop fabrication. In areas subject to periodic floods or storms, even the substructure should be designed to permit some shop precasting and quick field assembly.

Because heating and cooling increase the cost of concrete, designs with precast in lieu of cast-in-situ elements may be preferred.

In very cold climates, a warm working area is necessary to allow the work to proceed. Low productivity and poor quality of workmanship generally result from work done in unfavorable conditions.

4.5.2.2 Air-entrained concrete—Air entrainment is used to develop freeze-thaw durability in concrete. The requirements for entrained air and compressive strength should be specified separately. Mixes with entrained air having strengths up to 6000 psi (47.5 MPa) can be produced in most areas. However, the designer should verify the capability of the local producers.

4.5.3 Materials availability—Materials of adequate strength and quality may not be available locally.

Concrete strengths of 4000 to 5000 psi (27 to 34 MPa) using commonly available local aggregates are generally possible. With superplasticizers and pozzolans, 8000 to 9000 psi (55 to 62 MPa) can be produced with local materials in most areas. If it is necessary to bring in aggregates from other sources, the designer should evaluate the benefits gained by the additional cost.

High-quality, lightweight concrete aggregates, producing concrete weighing under 130 lb/ft³ (1760 kg/m³) and having

$f'_c = 3500$ psi (24.1 MPa), are not readily available at all locations. The designer should evaluate whether the benefits of this more expensive, specialized concrete outweighs its disadvantages.

4.5.4 Temporary foundations—The load-carrying capacity of the surface soils at the construction site should be considered. If the falsework has to be supported by driven piles, the cost of structures constructed on ground-supported falsework increases and precast options may be more economical.

Another consideration is the ability of the soils to support construction equipment needed in common construction operations: pile drivers, cranes, trucks, etc. In swampy or steep, rocky areas, expensive construction roads may have to be built. In this case an evaluation should be made of measures which could reduce pile-driving activities, crane use, and similar operations.

4.6—Environmental restrictions

Obtaining environmental clearance for a bridge project can be a complicated process affecting many facets of design and construction. This section deals only with those construction restrictions which might affect the design of a bridge.

4.6.1 Falsework—When the use of ground-supported falsework is prohibited, the restriction should be known early in the design process. It can then be handled routinely by selecting an appropriate type of structure. Troublesome cases arise when the restriction on use of falsework is not readily apparent in the design plans and is only discovered during the construction stage.

Use of falsework within a flood plain may be permitted only between specific dates, resulting in an artificially short construction season. For all practical purposes this may require a structure that does not need ground-supported falsework.

The requirements for falsework located next to railway tracks may be so restrictive as to make the use of such falsework very expensive and perhaps impractical. For example, steel posts set in holes filled with concrete and concrete crash walls may be required. As a result, a structure not requiring ground-supported falsework may be preferred for such locations.

4.6.2 Earthwork—Excavation, pile driving, and similar activities are usually prohibited during the spawning season, within streams and lakes used by game fish. The restriction generally lasts only a few months each year and is not a serious handicap for an average project. However, it should be considered in the context of the other constraints, so that the combined restrictions do not make the completion of the project impossible.

With current emphasis on preservation of environment and historical heritage, a restriction on construction of access roads is included in many contracts. In some cases, the restrictions are absolute, requiring the use of a cableway or a helicopter for construction of the intermediate supports and parts of the superstructure. Use of precast segments for both the superstructure and the piers is almost mandatory in such cases.

For construction within areas such as recreational parks, some agencies require immediate removal and disposal of excavated material. Such a requirement obviously favors

designs that minimize the foundation and earthwork needed in construction.

4.6.3 Construction—In many western states, fires during dry summer months are greatly restricted. On-site field welding is also prohibited during particularly dangerous forest fire periods. Such restrictions should be considered during the design phase when the structure type is selected, particularly if a steel structure seems to be an attractive alternative.

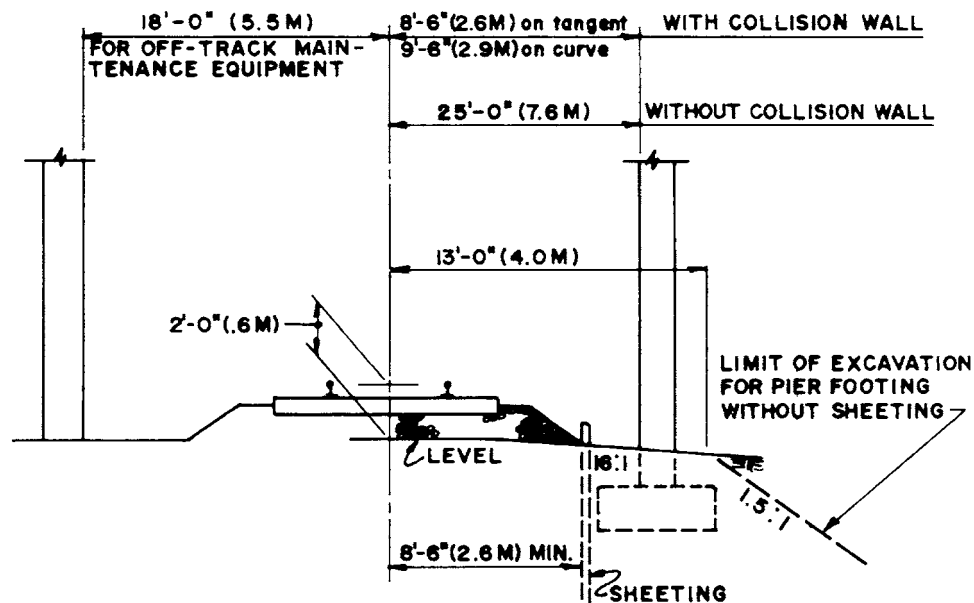
On construction projects close to sensitive areas, such as schools and hospitals, there are strict limitations as to the time of construction activity and amount of noise. The activity usually involved in such situations is pile driving. The remedy may be to use auger-cast piles or spread footings. The former may be very expensive, and the latter may lead to future problems of support settlements. On the other hand, a careful selection of working hours may alleviate the problem. Obviously, the problem is much more easily dealt with if it is considered during the design phase.

4.7—Maintenance of traffic

Traffic considerations are involved in construction of most bridges, though there are cases where traffic is entirely diverted from the construction site. The effects of traffic on construction will be particularly felt in construction clearances at the crossing. Traffic clearances for construction are generally smaller than those for permanent structures because of lower speed limits and the temporary nature of the clearances. In many instances, however, additional requirements imposed by the permitting agency may make the lesser construction clearances unattainable, thus affecting the layout and span lengths.

4.7.1 Railroad clearances—As an example of the process involved in determining the governing clearances, consider the clearance diagram shown in Fig. 4.7.1. The required final clearance from the center line of track to the face of a pier may be as small as 10 ft (3 m) for tangent tracks, but the closest the excavation for the foundation of the pier can be is 8 ft-6 in. (2.6 m), excluding the shoring and bracing. Thus the final clearance does not control the location of the pier and the designer should consider the size of footing, the depth of its location, the likely method of its construction, and the possibility of obtaining approval for a discretionary minimum clearance requirement.

4.7.2 Highway clearances—Prescribed temporary construction clearances often govern layout of spans. A typical example is the required vertical clearance over freeways in California, as shown in Table 4.7.2. The usual requirement is a clearance of 16 ft-6 in. (5 m) over the traveled way, but the temporary construction clearance may be as low as 14 ft-6 in. (4.4 m). However, for a structure constructed on ground-supported falsework, where a 40-ft (12.2-m) wide opening for traffic is needed, an adequate depth of falsework may be 2 ft-6 in. (0.75 m) to 3 ft-0 in. (1.0 m). This results in a final clearance of 17 ft-0 in. (5.1 m) to 17 ft-6 in. (5.3 m).



- * Excavations for pier footings, without shoring of track roadbed, should not be made closer than the limit line shown with slope of 1.6 to 1 starting at subgrade 13'-0" (4 m) from center line of track. Excavations closer than this sloped limit should not be undertaken without prior approval by the railroad of plans for shoring of track roadbed, and in any instance will not be approved closer than 8'-6" (2.6 m) horizontally from center line of track to near edge of excavation.

Note: This example is not applicable in all locations. Some railroads and states have different requirements and restrictions.

Fig. 4.7.1—Example of typical railroad clearance.⁴⁻²

Table 4.7.2—Example of falsework depth and span relationship⁴⁻²

Facility to be spanned	Minimum width of traffic opening, ft (m)	Opening width provides for	Required falsework span (b), ft (m)	Minimum depth required for falsework, ft (m)
Freeway	25 (7.6)	1 lane + (d)	32 (9.8)	1 ft 9 in. (0.53)
	37 (11.3)	2 lanes + (d)	44 (13.4)	2 ft 2 in. (0.66)
	49 (14.9)	3 lanes + (d)	56 (17.1)	2 ft 8 in. (0.81)
	61 (18.6)	4 lanes + (d)	68 (20.7)	3 ft 3 in. (1.0)
Nonfreeway	20 (6.1)	1 lane + (e)	27 (8.2)	1 ft 9 in. (0.53)
	32 (9.8)	2 lanes + (e)	39 (11.9)	1 ft 11 in. (0.58)
	40 (12.2)	2 lanes + (f)	47 (14.3)	2 ft 4 in. (0.71)
	53 (15.8)	3 lanes + (f)	59 (17.9)	2 ft 9 in. (0.84)
Special (a) roadways	64 (19.5)	4 lanes + (f)	71 (21.6)	3 ft 5 in. (1.04)
	20 (6.1)	1 lane + (e)	20 (6.1) (c)	1 ft 7 in. (0.48)
	32 (9.8)	2 lanes + (e)	32 (9.8) (c)	1 ft 9 in. (0.53)

(a) Uses such as fire or utility access or quasi-public road with very light traffic.

(b) Includes 7 ft (2.1 m) for two temporary guardrails.

(c) No temporary guard railing provided.

(d) 8 ft (2.4 m) and 5 ft (1.5 m) shoulders.

(e) 2 ft 4 in. (1.2 m) shoulders.

(f) 2 ft 8 in. (2.4 m) shoulders.

4.8—Project needs

Criteria governing the completion of an entire project may impose restrictive construction considerations on the design of an individual structure within the project. These considerations may involve a prescribed construction sequence (perhaps even an accelerated schedule), special construction loads, or uniformity of details and appearance.

4.8.1 Construction sequence—The requirement of allowing the passage of traffic through the construction site generally leads to a stage-constructed structure.

4.8.1.1 Partial-width construction—Bridge replacement projects usually require that the existing structure remain in service until a new one is completed, or that the new structure be constructed in the approximate location of the old one.

In such cases, about one-half of the new structure may be constructed adjacent to the old structure. After traffic is shifted to the completed half of the new structure, the old one is removed and the second half of the new structure is constructed. Typical construction considerations in such cases are:

- a. The partial-width structure should be stable at all times, either of itself or through use of temporary bracing
- b. The substructure of the new bridge should be of such a configuration as to permit half-width construction
- c. The deformations and creep shrinkage strains of the structure should be determined and provided for in the design and construction
- d. The part of the structure constructed first should be wide enough to accommodate the expected traffic
- e. If a portion of the existing structure is to be removed prior to the construction of the new structure, the amount of removal and staging of the removal should be determined by the design engineer to insure its stability
- f. The protection of the new structure during the removal of the old structure

4.8.1.2 Partial-length construction—In cases of multi-span bridges or multilevel interchanges, it may be necessary to construct the structure in stages, using either intermediate hinges or construction joints. A typical layout of a crossing over a highway, as shown in Fig. 4.1.2.2, may involve a span and a cantilever extending to some point into the adjacent span. These are constructed first, then the falsework is removed and the traffic shifted into the opening under the completed span. The remainder of the structure is then completed without interfering with rerouted traffic.

Typical construction considerations for such cases are:

- a. The initially constructed span may have to be designed as a simple span. It may be necessary to load the cantilever with temporary loads until the full dead load reaction is applied
- b. The camber for all affected spans should be calculated, based on the assumption of a reasonable construction sequence, taking into account creep and deflections due to all temporary loads. Improper camber may result in rough riding decks, poor drainage, and possibly a traffic hazard.

4.8.1.3 Detour bridges—In many cases, it may be possible to divert the traffic around the construction site by means of a detour. Sometimes an existing bridge at another location can be used. At other times it is necessary to construct a temporary bridge. Most agencies will impose the same design standards for temporary bridges as for permanent ones. However, the possibility should be thoroughly investigated, as substantial savings might be achieved by minor changes in the design requirements. Typically, the following construction considerations apply:

- a. Used materials may be permitted in the construction of a temporary bridge
- b. The contractor should be given complete latitude in selecting the basic material for the bridge (timber, steel, concrete)
- c. Use of prefabricated proprietary modular units should be allowed

4.8.2 Construction loads—For most structures, construction loads do not represent a major part of design loads. For structures constructed on falsework, all loads applied during construction are carried by the falsework. However, there are cases where construction loads represent major design loads and thus should be fully considered in the design.

4.8.2.1 Composite and segmental structures—Portions of these structures support all or part of their own formwork and construction loads. Also included are structures using a truss to construct some load-carrying elements that will later support the formwork and construction loads. Common construction considerations for these structures are the construction loads, their magnitude, and location. The structure may be designed to carry all the construction loads, or the contractor may be required to modify the original design for this purpose. Construction loads for structures using precast girders are usually smaller than the live loads and do not govern the design. Construction loads for segmental structures are quite large and generally control the design at various sections along the girder and at various times during erection. Requiring the contractor to redesign the structure for construction loads involves extensive engineering and expense. Furthermore, such contracts, being difficult to administer, can lead to disputes and litigation among the parties concerned, namely, the engineer, the owner, and the contractor. To avoid these possible difficulties, the designer may elect to estimate and design for the most likely construction loads. A conservative assumption may lead to some overdesign, but can avoid problems in the long run.

4.8.2.2 Earthmoving vehicles—On big freeway projects it may be necessary to move large quantities of embankment material from one side of the freeway to the other. Conventionally, one of the many overcrossing structures within the project is constructed first and used for moving the earth material by standard capacity dump trucks. However, an alternate approach used in some cases has proved to be fast and economical. One structure within the project may be designed for heavy earthmoving vehicles, with subsequent saving in cost of material moving. Fig. 4.8.2.2 shows typical axle loads for such vehicles.

The construction considerations for such cases may be:

- a. What is the size and weight of the heaviest construction vehicle likely to be used by the contractor?
- b. Is the provision for two-way traffic economically justifiable?
- c. Will the savings on the placement of embankment material outweigh the additional expense of the structure?

The answers to these questions should be evaluated by the design engineer working in cooperation with the owner.

A similar situation occurs when large amounts of earth should be moved from one side of a river crossing to the other.

4.8.3 Standardization—A uniform approach in the design of a number of structures within one larger project often results in an economy of the total project.

Generally each structure should be designed to fit a particular location; however, some economies may be realized by using the same type of structure for several locations.

Use of the same material in all the project structures is preferable. When materials of various strengths are available, only one or as few as possible should be used. The economics and the logic of specifying several strengths of concrete for a given project should be investigated and the decision made on merits of each individual case. To avoid placing errors, it is also preferable to use only one grade of reinforcing bars on a project.

4.9—Design of details

Generally, the economy of a structure is enhanced by striving for simplicity and repetitiveness in details, making each part of the structure easy to form, reinforce, fill with concrete, and strip.

4.9.1 Dimensions—Dimensions of elements should be large enough to accommodate reinforcement and permit placement and consolidation of concrete at the most congested point.

4.9.2 Repetitiveness—Forming of odd shapes is expensive. The multiple use of complicated formwork within a large project will help to reduce cost. Where voids are indicated, it is often advantageous to allow the contractor to substitute solid sections where structurally permissible. Giving the contractor a choice in the use of variations of details that do not materially affect the strength or the appearance of the structure is an effective way to increase the overall economy of a project. However, this permission should be given in the contract documents.

4.9.3 Slipforming—The overall economy of a project depends to a great degree on allowing the contractor the greatest possible freedom in selection of construction methods. Tall piers often can be constructed most economically by slipforming. It is difficult and expensive to slipform piers of a variable cross section. However, if the variability is restricted to one direction only, slipforming may become competitive.

4.9.4 Soffit lines—The top slab of box girders with widely spaced stems is usually of variable thickness. For the soffit line, a series of straight lines will function as well as a curve. Some saving may be achieved by giving the contractor a choice between using either a broken or a curved soffit line.

4.9.5 Placement of reinforcement—In cases where the reinforcement can be positioned a certain distance from the form face by the use of concrete or plastic blocks, there is usually no problem in maintaining proper cover. However, in cases where the reinforcement is next to a finished face (top reinforcement of slabs) or where the reinforcement is located inside a deep member, consideration should be given to some positive means of securing these bars in their proper places. In concrete slabs where workmen commonly walk on the top mat, a sufficient number of reasonably rigid bar chairs should be specified or detailed on the plans to insure that the constructed effective depth of the slab is as assumed in design. For bars located within deep members (e.g., two-layered arrangement of longitudinal reinforcement), additional supporting reinforcement should be detailed to fix their position. The cost of this auxiliary reinforcement is small compared to the cost of repair.

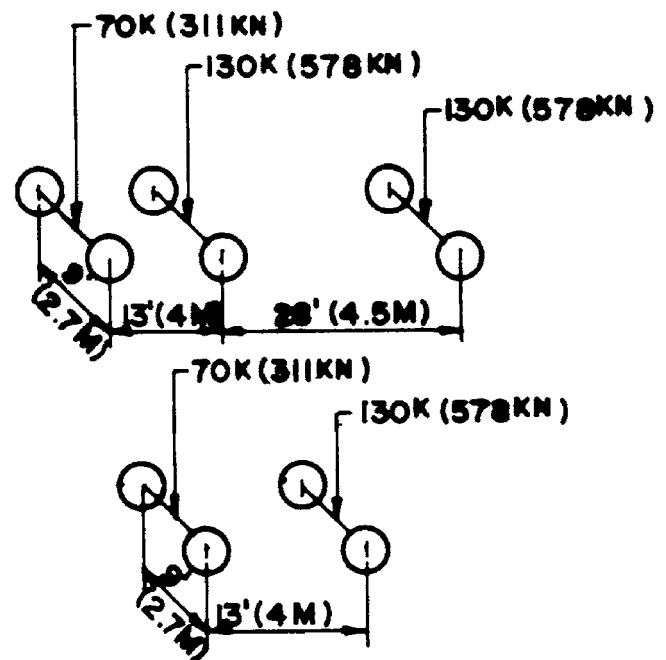


Fig. 4.8.2.2—Axle loads of typical earthmoving vehicles.

4.9.6 Placement of anchor bolts—In most instances where bolts are embedded in concrete, the placement demands a fairly high degree of accuracy. The simplest method of insuring the required accuracy involves the use of a prefabricated steel template/spacer to which the bolt heads are tack welded (incidentally, care shall be exercised to avoid welding on any other part of the bolt). The template is wired to adjacent reinforcement (if the reinforcing is too far away, additional reinforcement should be called for) and can be positioned accurately and kept in place during placement of concrete.

Some agencies require that the anchor bolts be set in holes drilled after the concrete has hardened. In this case the designer should space the reinforcement to provide ample tolerance to clear the drilled hole.

4.9.7 Hinges—The term “hinge” as used in the discussion that follows denotes a joint located within a structural member and constructed in such a way as to permit the assumption of no moment at the joint while allowing transfer of vertical and/or horizontal shear.

In terms of maintenance problems, hinges have been found to be particularly troublesome. These problems are caused by a combination of factors, some originating in design and others in common construction practices. In the past, there has been some difficulty in assessing all the forces acting upon various parts of a common hinge. Most hinges, even with the best expansion bearings, involve axial tension in the direction of the girders. ACI 318, Section 11.9, Special Provisions for Brackets and Corbels, is quite helpful in clarifying this point.

Another point to recognize is that the ever-present “bump” at the hinge location can become quite large due to settlements, creep deformations, and deterioration. This can cause impact forces much greater than those used in design for conventionally reinforced structures. The combination of high

bending and shear stresses at most hinges requires placement of large amounts of horizontal and vertical shear reinforcement in addition to tensile reinforcement. This congestion of reinforcement makes the placement and consolidation of concrete difficult, and often results in rock pockets and honeycombs. Adequate proportioning of sections and proper supervision of construction are necessary. In many cases it may be advantageous to replace some of the conventional mild steel reinforcement with short post-tensioning tendons. The post-tensioning systems used should be those that allow stressing and anchorage of short tendons without seating loss.

A similar instance of theoretical hinges at bottoms (or tops) of columns needs careful attention to detail. For initially applied loads (dead loads) and depending on the care used in construction, these hinges will have almost no moment. However, after the application of large axial loads, and after some deterioration of the hinge materials takes place (e.g., expansion joint material compresses and becomes hard), the moments being transferred across such hinges can become quite large. Generally, the stresses induced at the hinge itself are rarely objectionable; however, at other joints of the structure, the additional moments and stresses caused by the partial fixity of the hinges may be greater than anticipated in the design.

4.9.8 Fixed end supports—In its pure form, this type of support is seldom used because full fixity cannot be achieved. The designer needs to review the design assumptions very carefully, particularly in view of likely location of construction joints and number of reinforcement splices. A 100 percent fixity is probably not a justifiable assumption, and therefore some increase in adjacent positive moments in design should be provided.

4.9.9 Accuracy of construction—The design assumptions and design methods do provide some consideration of the inaccuracy of construction. The phi factor used in load factor design does take into account some variability of quality of material, and the column design criteria assume a certain eccentricity of loads.

The accuracy of construction to be considered here deals with cases arising out of construction methods and procedures beyond these allowances that need to be considered by the designer.

4.9.9.1 Pile location—All pile footings should be large enough to accommodate the piles that may be driven or drilled out of exact location. The design should provide for a tolerance of 3 in. (75 mm) out of location for the pile group. A 3-ft (1.0-m) c. to c. minimum pile spacing and a 1.5-ft (0.5-m) minimum edge distance are considered normal for 12-in. (300-mm) piles. Where a greater accuracy is needed (such as single-line pile bents consisting of driven or drilled piles and pile extensions above original ground), a pile-locating template should be used.

The designer is cautioned that a pile group having less than a 3-in. (75-mm) eccentricity may have within it piles located 6 in. (150 mm) or more out of location. The ACI 318, Section 15.5 provision for reduction of shear if the pile center lies within one-half the pile diameter of the $d/2$ distance from the column face should be used with judgment.

4.9.9.2 Precast elements—Precast segments have been used to assemble I-girders, box girders, slabs, whole superstructures, piers, columns, abutments, and walls. With use of match casting, there is a reasonable fit between the individual segments. Problems usually occur with longer girders (or higher piers) where small inaccuracies may add up to a sizable deviation from anticipated alignment. For girders, this may produce unsightly vertical sag or lateral wobble; piers and columns may have a bowed shape or be out of plumb in any direction.

The task of the designer is to provide for this eventuality by incorporating some method of correction into the design. For sagging girders it may be a provision for additional ducts for corrective post-tensioning, as well as an occasional cast-in-place segment. Cast-in-place segments also may be used for correcting alignment of piers and columns. Thin shims in conjunction with epoxy mortar also have been used for this purpose.

The design of segmental precast structures or parts of structures should readily permit adjustments during construction.

4.10—Selection of structure type

When several types of structure seem to be equally economical and suitable for a particular location, the decision may be deferred to the bidding process through preparation of alternative designs. Obviously, the additional expense of preparing two or more designs has to be justified by the possible savings. Such procedures are usually reserved for major structures. The two main requirements for such alternative designs—the equivalency of strength and appearance—are not easily met unless the intent is accurately described or illustrated. The additional time needed for review of such alternative designs also complicates the process.

The following sections outline some of the construction considerations connected with various types of concrete structures:

4.10.1 Concrete slab bridges—A concrete slab bridge is the simplest and least expensive structure that can be built within the span limitations for this type of superstructure. It can be conventionally reinforced, pretensioned or post-tensioned. It can be built on ground-supported falsework or constructed of precast elements.

4.10.1.1 Cast-in-place bridges—The simplicity of the concrete slab bridge may lead to overconfidence on the part of the contractor, often resulting in poor workmanship. Falsework founded on soft or shifting soils has led to differential settlements and bulges in the soffit line during construction. The solution may be to specify allowable soil-bearing values for falsework design or to require use of pile-supported falsework.

Another problem often encountered during construction of slab bridges is the difficulty of keeping the top mat of reinforcing bars in its proper position. Since the workmen usually walk on these bars, supports have to carry construction loads, as well as the weight of the bars. If the size and the spacing of the bar chairs are not shown on the plans, only the minimal amount will be used during construction, with undesirable consequences.

The designer, therefore, should make sure that a sufficient number of reasonably rigid bar chairs is called for on the plans to keep the top mat of reinforcement in its place.

4.10.1.2 Precast slab bridges—Precast slab bridges constructed as simple spans often exhibit poor riding qualities due to the discrepancy between the camber and actual deflections. Transverse joints at piers further impair the riding quality of the deck and often present serious maintenance problems. Providing reinforcement in the topping slab to develop continuity can alleviate this problem.

Precast slab bridges constructed with unreinforced grout keys in the longitudinal joints, between individual units, also develop maintenance problems. The grout keys often fail, allowing differential live load deflections between units. An asphalt concrete overlay does not solve the problem, since the cracks usually continue to show up through repeated overlays.

A reinforced concrete key or other positive connection between individual precast units, such as transverse post-tensioning, is helpful.

4.10.2 Reinforced concrete T-beams—This type of structure is usually constructed on ground-supported falsework. It is most suitable for bridges of short span lengths of 30 to 80 ft (9 to 24 m).

4.10.2.1 Reinforcement—Due to the narrow stem width, a multilayer arrangement of reinforcement in the positive moment region is required. This makes concrete placement difficult, often resulting in rock pockets. The selected width should accommodate the reinforcement while allowing easy placement and consolidation of the concrete.

The problem of maintaining the top mat of reinforcement in its proper position, as discussed in Section 4.10.1.1, is even more serious for thin slabs where a shift of reinforcement by 1/2 in. (12 mm) in a 7-in. (178-mm) slab will mean a change in effective depth of 10 percent.

Girder stirrups and bent-up slab reinforcement bars do provide some support for the top mat; however, the designer needs to be sure that the supports provided will prevent a downward displacement of the reinforcement under the probable construction loads.

4.10.2.2 Construction joints—Usually the T-beam superstructure is constructed in two separate stages: the stems and the slabs. To minimize cracks in the tops of stems due to temperature and shrinkage stresses, as well as possible differential settlement of falsework, some longitudinal reinforcement should be placed within the stems just below the construction joint.

4.10.2.3 Longitudinal joints—For very wide structures constructed full width, concrete placement and finishing becomes difficult and expensive. Temperature stresses and movements imposed on the substructure become rather large. For these reasons, a longitudinal joint within the structure becomes almost a necessity for bridges wider than about 60 to 70 ft (18 to 21 m).

A typical detail in such cases may involve: a construction joint, or an expansion joint, or a construction joint and an expansion joint with a closure slab as shown in Fig. 4.10.2.3.

The decision as to which detail to use will depend on the total width of structure; the magnitude of initial deflection

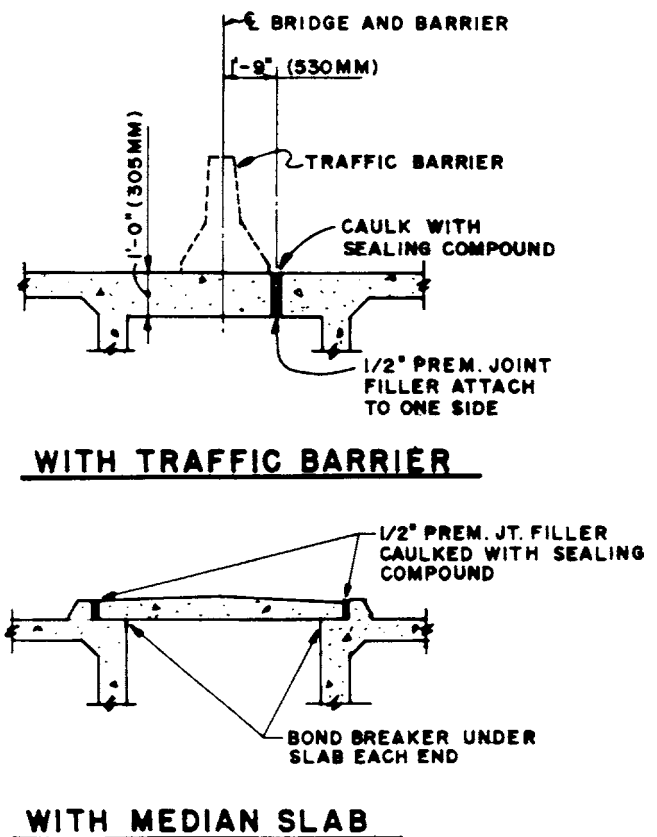


Fig. 4.10.2.3—Examples of longitudinal expansion joints for wide structures.

and rotation to be accommodated by the joint; the amount of creep deflection accumulated by the first half before the second half is constructed; and finally by the need to provide for maintenance of traffic.

To minimize confusion of the drivers, the joint should be located on a lane line or within a median.

4.10.3 Precast, prestressed girders—These are most suitable for locations where the use of falsework is either prohibited, impractical, or too expensive. The construction time is usually shorter than that needed for cast-in-place girders. The girders are designed to carry dead load and construction loads as simple span units. Live load and superimposed dead load design may or may not use continuity and composite action with the cast-in-place slab.

The considerations mentioned for T-beams also apply to wide structures constructed with precast prestressed girders. The discussion of possible downward displacement of reinforcement during concrete placement mentioned in Section 4.10. also applies.

4.10.3.1 Transportation and handling—Most state highway departments require a transportation permit for any load over 80 ft (24 m) long, and many will not issue permits for loads over 100 ft (30 m) in length. Thus, long girders may have to be brought to the site in segments and assembled there, resulting in increased cost of field work. Poor access roads can also require segmental construction.

Long, precast, prestressed girders are usually laterally unstable (particularly if the compression flange is

narrow) until incorporated into the structure. Firm ground is needed to store the girders, as well as to support the required lifting cranes. Pickup points should be determined beforehand and lifting hooks installed. Once erected, and until the diaphragms are cast, the girders should be braced.

4.10.3.2 Camber—Multispan simple span bridges using precast, prestressed girders do not usually provide a comfortable ride. Some of the steps that can be taken to minimize this problem include:

- a. Make girders continuous for live loading by casting concrete between ends of girders
- b. Use a liberal depth for the girders
- c. Specify a maximum length of time that the girders may be stored without being loaded with at least some dead load

4.10.4 Nonprestressed reinforced concrete box girders—This type of structure is adaptable for use in many locations. It has been used in both metropolitan and rural areas, for single structures, or for entire interchanges. The popularity of this type of structure lies in its economy for a wide range of spans and layouts and its reasonably good appearance.

4.10.4.1 Reinforcement—The stem thickness selected should accommodate the reinforcement and allow proper placement and consolidation of concrete. Particularly in the case of boxes over 7 ft (2.13 m) deep, a stem thickness larger than the frequently used minimum of 8 in. (200 mm) should be considered.

The discussion of possible downward displacement of reinforcement during concrete placement mentioned in Section 4.10.2.1 is equally valid here.

4.10.4.2 Construction joints—Temperature and shrinkage stresses, as well as possible settlement of falsework after the initial placement of concrete, often result in vertical cracks in the webs. Additional horizontal reinforcement in the webs near the construction joint has been found helpful in restricting the size of these cracks.

4.10.5 Post-tensioned concrete box girders—Post-tensioned box girder construction affords many advantages in terms of safety, appearance, maintenance, and economy. Long spans may be constructed economically, thereby reducing the number of piers and eliminating shoulder obstacles at overpasses. Obstacle elimination greatly enhances the recovery area for out-of-control vehicles.

4.10.5.1 Falsework—Construction methods for building post-tensioned concrete box girder bridges are many and varied. Construction on ground-supported falsework is prevalent in the western part of the United States. In recent years this method has gained acceptance in the eastern and midwestern states. The use of ground-supported falsework construction will increase as projects are evaluated to determine the most economical construction method.

Falsework construction methods may be modified when site restrictions dictate the elimination of ground-supported falsework. The falsework may be supported by an auxiliary truss spanning from pier to pier. An example of this method of construction is the Denny Creek Bridge (Reference 4-3) located in the Snoqualmie National Forest east of Seattle, Washington. The extreme ecological sensitivity of the area required an esthetically pleasing structure with minimal

damage to the environment. To satisfy the restrictions, the winning contractor chose to cast the box girder section in forms, supported by a traveling truss. The truss, 330 ft (100 m) long and weighing 540 tons (4800 kN), moved forward span by span as construction progressed. Denny Creek Bridge was the first instance of the use of this system in the United States.

Another modification of ground-supported falsework occurred on the Napa River Bridge⁴⁻⁴ in California. Falsework towers at 70-ft (21-m) intervals with wide flange girders were used to support cast-in-place box girder cantilevers. The reuse of the towers and forms had an economic advantage over conventional construction. In addition, a required 70-ft (21-m) navigation channel was maintained during construction.

These are only a few of the possible modifications of falsework construction; many others have been successfully employed and new ones will be developed. Possible restrictions on each project should be thoroughly investigated so the most economical method of construction can be used.

4.10.5.2 Construction options—Post-tensioned box girder construction offers many options to both designers and builders for a successful project at minimum cost. The size of components cast at one time may be varied from partial spans or partial cross sections to entire spans. The components may also be precast and assembled with the post-tensioning tendons. The prestressing tendons, whether pretensioned or post-tensioned, can be located to place the required forces in the most efficient manner. The tendons may be straight, draped, continuous throughout the structure, or of partial lengths. For instance, the contractor on the Napa River Bridge used a longitudinal loop tendon system. The 12-strand tendons extended from the end of one cantilever to the other cantilever, where they looped around and returned.

Partial stressing of post-tensioning tendons may control shrinkage and temperature cracks until the concrete gains sufficient strength to allow full prestress application. Proper analysis of the structure and the use of some of these options, either alone or in combination, will result in cost savings while providing structural integrity.

4.10.6 Post-tensioned segmental construction^{4-5,4-6}—Since about 1970, when this European technology was introduced into the United States, segmental construction has added a new dimension to the design and construction of post-tensioned concrete box girder bridges. The basic concept is to provide cost saving through standardization of details and multiple use of construction equipment.

Segmental concrete bridges may be precast by either the long line or short line system. The segments may also be cast-in-place. They may be erected by the method of balanced cantilevers, by progressive placement span by span, or by launching the spans from one end. Almost any combination may be used to provide the most cost-efficient use of materials, labor, and equipment. Both the designer and contractor should have the opportunity to evaluate and choose the proper combination.

4.10.6.1 Standardization—Standardization is accomplished by varying the details as little as possible. The spans may be sloped or vertical, but preferably should remain a

constant thickness for the entire length of the structure. Ideally the only variation would be the spacing of the shear reinforcement.

Most existing projects have incorporated sloped webs. Many believe sloped webs are esthetically pleasing; as a practical matter they facilitate form stripping. However, engineers should consider the possible problems encountered by sloping the webs of variable depth box girders. A constant web slope means the width of the bottom slab decreases as the depth of the section increases. The forms should be fabricated to accommodate the variable dimensions. Sometimes the saving in substructure quantities due to reduced superstructure weight justifies the additional form cost.

The top slab should remain of constant cross section. An exception occurs when minor grade variations are accomplished by increasing or decreasing the top slab thickness. Any variations in the slab thickness should be confined to the top of the slab to eliminate modification of the inside forms.

The top slab may be of variable depth in the transverse direction. The design of the box girder section in the transverse direction is based on rigid frame analysis requiring a moment-carrying capability at the joints. The thickening of the joint area where the webs connect to the slab provides the moment capacity along with extra concrete to carry the shear stress occurring between the web and slab.⁴⁻⁷

Transverse prestressing of top slabs of 40 ft (12 m) or greater width, in conjunction with variable depth, proves especially advantageous from the viewpoint of economics and durability. If the transverse tendons are located close to the centerline of the slab at the ends of the cantilevers and extend in a straight line through the slab, the prestressing force will be in the correct position, relative to the neutral axis, to carry both the negative and positive moments. This of course assumes a normal crown. The same will not be true for segments with constant superelevation or with superelevation transition.

The bottom slab should have a constant thickness in both directions throughout, with two exceptions. The bottom slab may have to be thickened in the transverse direction at the web junctions for the previously mentioned reasons. Also, if the segments are erected by balanced cantilevers, the bottom slab generally has to be thickened over the piers to carry the compressive stresses. However, the slab should be back to normal thickness within two or three segments away from the pier to return to normal casting procedures as quickly as possible and to minimize dead load.

4.10.6.2 Construction loads—The concept of construction loads having an influence on design as discussed in Section 4.3 becomes very important in segmental concrete design and construction. The amount of prestressing required in some areas may well be governed by construction loads. In other areas, service loads may govern. For the design to be complete, all loads should be considered even though the exact location and magnitude of the construction loads may not be known.

4.10.6.3 Design procedure—The proper design procedure is to investigate methods available to construct the bridge and to base the design on the most probable and the

most economical method, recognizing the pertinent restrictions. Not only the method of construction, but the time or rate of construction, are important because of the many time-dependent factors (such as concrete creep and shrinkage) that can have significant effects on stresses and deformations. The designer should consider details of the assumed construction method, including locations and magnitudes of construction loads when preparing the plans and specifications. The project may then be opened up to alternative methods of construction consistent with the restrictions and with the stipulation that allowable stresses are not to be exceeded, including secondary and time-dependent effects. If construction loads are exceeded, or the timing is significantly different, it will be necessary for the contractor to provide a redesign.

In this way the contractor has the opportunity to develop the most economical method. Three situations can occur:

- a. The contractor can agree with the designer's assumptions and build the bridge according to the plans and specifications.
- b. The contractor can develop an economical construction scheme with smaller construction loads and redesign the bridge to save material and thereby produce some cost saving.
- c. The contractor can develop a more economical construction scheme with larger construction loads and redesign the bridge to provide some cost saving.

This system has some minor drawbacks. The projected cost savings should exceed the redesign costs by a significant amount to make the effort worthwhile. Included in the redesign costs should be the engineering costs involved in reviewing and approving redesigns. However, engineering costs are usually low when compared with construction costs on a project. Contractors have to spend money in preliminary designs to verify methods without any assurance of getting the work. Unsuccessful bidders probably will pass this cost on to subsequent projects.

The real advantage of this system is that it provides the contractor with an opportunity to bid the method most suitable to the contractor's expertise. Therefore, the cost saving is obtained through closer competition.

4.11—Construction problems^{4-8,4-9}

Prevention of the following examples of construction problems that have been encountered on concrete bridge projects should be considered during design.

4.11.1 Cracking due to shrinkage and creep—The subject of shrinkage and creep is quite complicated and involves a large number of factors, some easy to calculate and others difficult to predict. These factors include: strength of concrete, age at which the concrete will be subjected to stress, magnitude of maximum stresses, temperature variations, quality of concrete, quality of curing, and general quality of construction.

The designer works with a set of well defined design specifications and design procedures. The product is expected to be a properly designed structure that can be constructed and put into service on time, within budget, and without any

major problems. The problems that develop usually come from factors that are at variance with the assumptions used in design. These variations can arise from changes in weather, variability of construction materials, and nonuniformity of workmanship. There are cases where rates of shrinkage have varied by as much as 100 percent from one batch of concrete to another on the same project. Similarly, variations of 50 percent in creep rates are not uncommon. Therefore, it is prudent and cost-effective for the designer to consider several rates of creep and shrinkage and to assess the probability of their occurrence and their effects on the structure, versus the cost of mitigating measures.

Some measures that have been used to reduce the undesirable effects of excessive shrinkage rates are: stage construction of long structures, selective use of high cement content, concrete additives, and liberal use of continuous nonprestressed reinforcement.

Another common reason for variations in creep rates is the discrepancy between assumed and actual concrete strength at the time of load application. This is particularly true in case of loads applied at a later time in the age of concrete; e.g., railings, curbs, overlays.

4.11.2 Temperature cracking—This problem often manifests itself on box girders where a very thick bottom slab is used to provide the necessary compressive area at the supports. Heat is generated by the hydration of the cement, with additional input from hot weather in some instances. The remedies to be considered during the design stage might be:

- a. Reduction of slab thickness (increased depth of structure)
- b. More longitudinal reinforcement within the section
- c. Reduction of applied moments by use of temporary supports, stays and tendons

The designer might also specify:

- d. A reduction in rate of concrete placement
- e. A reduction in cement content
- f. The use of retarders and/or pozzolans in the mix
- g. Cooling of mixing water and aggregates

In cases where the outdoor temperature represents a large part of the heat input, use of insulated forms should be considered. As always, the provision of required minimum ratios for temperature reinforcement should not be neglected.

4.11.3 Cracking due to tendons—Cracking of concrete along planes tangential to tendons has been observed on several structures. This cracking is the result of stresses exerted by the tendons on the concrete at locations where there is change in tendon alignment, whether intentional (as in curved girders, or in girders with draped tendons, etc.) or accidental (insufficient supports for tendons or workers stepping on them).

In cases of intentional curvature, the forces can be calculated and reinforcing provided as needed for tension in the plane normal to the tendons. In cases of unintentional curvature or kinks in the tendon path, the designer should anticipate and estimate not only in which direction the cracking may occur, but should also evaluate the cost of additional reinforcement versus the cost of better inspection. The use of rigid ducts and close spacing of duct supports is quite helpful.

4.11.4 Crushing of ducts—This problem occurs on post-tensioned structures in areas of duct curvature where ducts are closely placed or actually bundled. As a result, there is only a thin layer of concrete (sometimes none) to resist the radial tendon forces. This problem has surfaced on curved girders, as well as on girders with tendons draped in the vertical plane. The following remedies are available to the designer for consideration in these situations:

- a. The designer should be satisfied that the concrete section is large enough to accommodate the required reinforcement and ducts. “Large enough” is intended to mean that there is enough concrete around each duct to resist and distribute, without distress, the horizontal and/or vertical shears imposed by the tendons on the concrete. In practical terms, it means that the designer should select a comfortable depth for the structure, sufficient width for the girders, and reasonable spacing for the diaphragms.
- b. In cases of unavoidably tight tendon spacing, the designer should consider specifying metal saddles or spacers between the ducts. Metal duct saddles have been used on some projects and are available commercially.
- c. In some cases where it is too late for the other remedies, a carefully thought out post-tensioning sequence will help, although duct crushing may still occur. Grouting of such ducts will be difficult and may result in incompletely grouted ducts.

4.11.5 Construction joints—Almost all structures are designed on the assumption of a monolithic construction. However, it is very seldom that a structure can be constructed in such a manner; most structures have a number of construction joints and some (such as segmental bridges) have many of them. In situations where there will be a large number of construction joints within a structural member, there is normally no need to specify their location; however, in cases of only a few construction joints in a span or a pier, it is best to limit their location. Construction considerations in connection with construction joints are basically the need to assure a monolithic behavior of the structure, despite the presence of some degree of discontinuity represented by the construction joint. Continuity is achieved by insuring the transfer of joint forces (axial forces, moments, and shears) across the joint without distress.

4.11.5.1 Joint location—In cases where only a few joints are used, the preferred location in the span is at or near an inflection point. Here the moment is near zero, the shears are lower, and deflection is minimal—conditions favorable for a distress-free construction joint.

In cases where many construction joints are used in a span (segmental construction), the location of the joints depends on the construction method and available equipment and is usually not specified. Where joints are exposed, regularity of spacing and uniformity of appearance make the best impression and thus should be specified.

4.11.5.2 Joint configuration—Transfer of axial forces across the construction joint usually presents no problem as long as the joint surface is normal to the direction of forces and the