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24

Bridge Decks and Approach Slabs

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24.1 Introduction

This chapter discusses bridge decks and structure approach slabs, the structural riding surface that typically is the responsibility of bridge design engineers when developing contract plans and details. Decks and approach slabs are usually not specially designed for each bridge project, but are instead taken from tables and standard plans either developed and provided by the owner or approved for use by the owner. However, as the load and resistance factor design (LRFD) is adopted by various agencies, standards that were developed previously must be reviewed and revised to comply with these new standards.

Not only do decks and approach slabs provide the riding surface for vehicular traffic, but they also serve several structural purposes. The bridge deck distributes the vehicular wheel loads to the girders, which are the primary load-carrying members on a bridge superstructure. The deck is often composite with the main girders and, with reinforcement distributed in the effective regions of the deck, serves to impart flexural strength and torsional rigidity to the bridge. The structure approach slab is a transitional structure between the bridge, which has relatively little settlement, and the roadway approach, which is subject to varying levels of approach settlement, sometimes significant. The approach slab serves as a bridge between the roadway and the primary bridge and is intended to reduce the annoying and sometimes unsafe “bump” that is often felt when approaching and leaving a bridge.

24.2 Bridge Decks

There are several different types of bridge decks, with the most common being cast-in-place reinforced concrete [1]. Other alternative deck types include precast deck panels, prestressed cast-in-place decks, post-tensioned concrete panels, filled and unfilled steel grid, steel orthotropic decks, and timber. These less common types may be used when considering deck rebar corrosion, deck replacement, traffic, maintenance, bridge weight, aesthetics, and life cycle costs, among other reasons. This chapter will emphasize cast-in-place reinforced concrete decks, including a design example. The alternative deck types will be discussed in less detail later in the chapter.

24.2.1 Cast-in-Place Reinforced Concrete

The extensive use of cast-in-place concrete bridge decks is due to several reasons including cost, acceptable skid resistance, and commonly available materials and contractors to do the work. Despite these advantages, this type of deck is not without disadvantages.

The most serious drawback is the tendency of the deck rebar to corrode. Deicing salts used in regions that must contend with snow and ice have problems associated with corrosion of the rebar in the deck. In these areas, cracks caused by corrosion are aggravated by the results of freeze–thaw action. Damage due to rebar corrosion often results in the cost and inconvenience to the traveling public of replacing the deck. In an effort to minimize this problem, concrete cover can be increased over the rebar, sealants can be placed on the deck, and epoxy-coated or galvanized rebar can be used in the top mat of deck steel. However, these solutions do not prevent the deck from cracking, which initiates the damage.

A common means of reducing deck rebar corrosion is the use of coated rebar. One drawback to this type of rebar is that it is often difficult to protect epoxy-coated and galvanized rebar during construction. Small nicks to the rebar, common when using normal construction methods, may be repaired in the field if they are detected. However, it is difficult to make repairs that are as good as the original coatings.

Despite corrosion problems associated with a cast-in-place reinforced bridge deck, it continues to be the most common type of deck built, and therefore a design example will be included in Section 24.2.1.3.

24.2.1.1 Traditional Design Method

Traditionally, cast-in-place bridge decks are designed assuming that the bridge deck is a continuous beam spanning across the girders, which are assumed to be unyielding supports. Although it is known that the girders do indeed deflect, it greatly simplifies the analysis to assume they do not. By using this method the maximum moments are determined and the deck is designed. This design method, in which the deck is designed as a series of strips transverse to the girders, is referred to as the "approximate strip design method." This method has been refined over time and has now been adapted to LRFD in the 1994 AASHTO-LRFD Specifications [2]. All references to this code will be shown in brackets.

24.2.1.2 Empirical Design Method

More recently, an alternative method of isotropic bridge deck design has been developed for cast-in-place concrete bridge decks in which it is assumed instead that the deck resists the loads using an arching effect between the girders. The 1994 AASHTO-LRFD Specification includes an empirical design method for decks using isotropic reinforcement based on these arch design principles [9.7.2]. Under this method no analysis is required. Instead, four layers of reinforcement are placed with no differentiation between the transverse and longitudinal direction. However, to use this method the conditions outlined under [9.7.2.4] must be satisfied. These conditions include requirements for the effective length between girders, the depth of the slab, the length of the deck overhang, and the specified concrete strength.

The deck overhang itself is not designed using this method. Instead cantilever overhangs are designed using the approximate strip method described above and used in the design example below.

24.2.1.3 Design Example

Given

Consider a cast-in-place conventionally reinforced bridge deck (Figure 24.1). The superstructure has six girders spaced at 2050 mm. The deck width is 11,890 mm wide and the overhang beyond

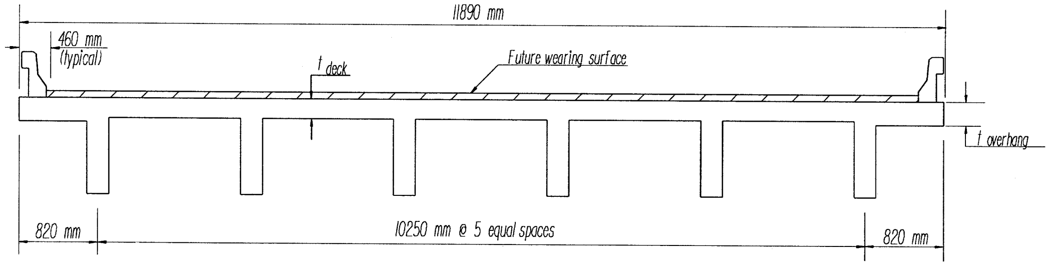

FIGURE 24.1 Typical section.

the exterior girder is 820 mm from the girder centerline. The unit weight of concrete is assumed to be 23.5 kN/m^3.

Find

Based on the 1994 AASHTO-LRFD Specifications, use the Approximate Strip Design Method for Decks [4.6.2.1] to determine the deck thickness, design moments, and the detailing requirements necessary to design the bridge deck reinforcement.

Solution:

1. **Determine the Deck Thickness** [Table 2.5.2.6.3-1] [9.7.1.1]

$$t_{deck} = (S + 3000)/30 = (2050 + 3000)/30 = 168.3 \text{ mm}$$

where S = the girder spacing. The minimum required deck thickness, excluding provisions for grinding, grooving, and sacrificial surface is $t_{\text{deck}} = 175$ mm. ⇐ controls

The deck overhang is often a different thickness than the deck thickness. This may be for aesthetic, structural, or other reasons. For this example, assume the deck overhang is a constant thickness of $t_{\text{overhang}} = 250$ mm.

2. **Determine Unfactored Dead Loads**
For simplicity the deck will be designed as a 1-m-wide one-way slab. Therefore, all loads will be determined on a per meter width.

Slab:	q_{DS}	$= (23.5 \text{ kN/m}^3)(0.175 \text{ m})(1 \text{ m}) = 4.11 \times 10^{-3}$ kN/m
Overhang:	q_{DO}	$= (23.5 \text{ kN/m}^3)(0.250 \text{ m})(1 \text{ m}) = 5.88 \times 10^{-3}$ kN/m
Barrier rail:	P_{DB}	= 5.8 kN per 1 m width
Wearing surface:	q_{DW}	$= (1.70 \text{ kN/m}^2)(1 \text{ m}) = 1.70$ kN/m

3. **Determine Unfactored Live Loads** [4.6.2.1] [3.6.1.3.3] [3.6.1.2.2]
   a. *Wheel load*:

Truck axle load = 145 kN/axle.

The axle load of 145 kN is distributed equally such that each wheel load is 72.5 kN. These 72.5-kN wheel loads are moved within each lane, with an edge spacing within the lane of 0.6 m, except at the deck overhang where the edge spacing is 0.3 m as specified in [Figure 3.6.1.2.2-1].

   b. *Calculate the number of live load lanes* [3.6.1.1.1]:
Assume for this example the barrier rail width is 460 mm, which implies that the clear distance between the face of rail w = 11,890 mm – 2(460 mm) = 10,970 mm. Therefore, the number of lanes is N = (10,970)/3600 = 3.05, i.e., 3 using just the integer portion of the solution as required.

TABLE 24.1 Wheel Load Layout

Alternative	Wheel Load Layout (Span/Distance from left end of span)
M_{+ve} Alternative 1 (one lane)	Span 2/820 mm
	Span 3/570 mm
M_{+ve} Alternative 2 (one lane)	Span 2/1030 mm
	Span 3/780 mm
M_{+ve} Alternative 3 (two lanes)	Span 2/820 mm
	Span 3/570 mm
	Span 4/820 mm
	Span 5/570 mm
M_{-ve} Alternative 1 (one lane)	Span 2/1150 mm
	Span 3/900 mm

c. *Determine the wheel load distribution* [4.6.2.1.6] [Table 4.6.2.1.3-1] [4.6.2.1.3]:
For cast-in-place concrete decks the strip width, w_{strip}, is
Overhangs:

$$w_{\text{strip}} = 1140 + 0.833X$$

where X = the distance from the location of the load to the centerline of the support. If it is assumed that the wheel load is pushed as close to the face of the barrier as permitted by the Code, i.e., 300 mm:

$$X = 820 \text{ mm} - 460 \text{ mm} - 300 \text{ mm} = 60 \text{ mm}$$

Therefore, for the overhang $w_{\text{strip}} = 1140 + 0.833(60) = 1190$ mm.
Interior slab:

$$\text{Positive moment } (M_{+ve})\text{:} w_{\text{strip}} = 660 + 0.55S = 660 + 0.55(2050) = 1788 \text{ mm}$$

$$\text{Negative moment } (M_{-ve})\text{:} w_{\text{strip}} = 1220 + 0.25S = 1220 + 0.25(2050) = 1733 \text{ mm}$$

where S = the girder spacing of 2050 mm.

d. *Determine the live loads on a 1-m strip*:
The unfactored wheel loads placed on a 1-m strip are

Overhang: 72.5 kN/1.190 m = 60.924 kN/m

Positive moment: 72.5 kN/1.788 m = 40.548 kN/m

Negative moment: 72.5 kN/1.733 m = 41.835 kN/m

4. Determine the Wheel Load Location to Maximize the Live-Load Moments

Three alternative wheel load layouts will be considered to determine the maximum positive moment for live load. Three alternatives are investigated to illustrate the method used to determine the maximum moments. Only the controlling location of the wheel loads will be used to determine the maximum factored negative moment. In Table 24.1, deck spans are numbered from left to right, with the left cantilever being span 1. Distances are measured from the leftmost girder of the span. Wheel loads are placed at the locations listed in the table.

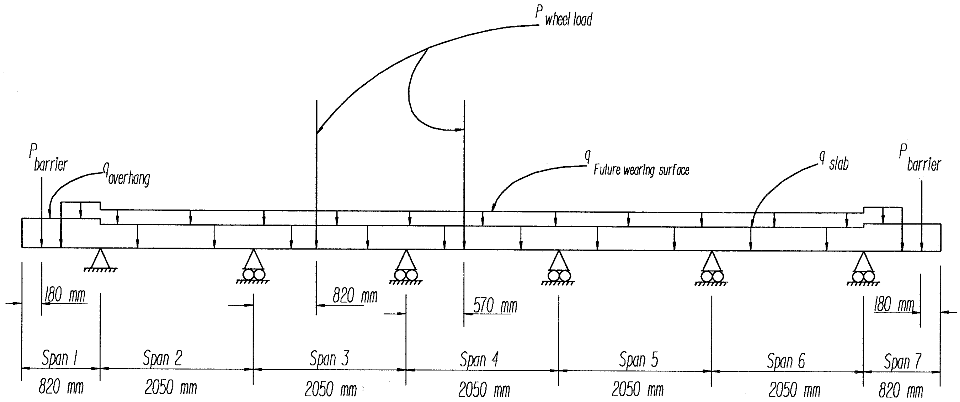

FIGURE 24.2 Loads: M_{+ve} Alternative 1 (one lane).

5. **Calculate Unfactored Moments**
   Apply the unfactored loads determined in the steps above to a continuous 1-m-wide beam spanning across the girders (Figure 24.2). Based on these loads, the unfactored moments for the dead load and the live load alternatives are listed in Table 24.2. The bridge is symmetrical about the end of span 3; thus loads are only given for the left half of the bridge. The locations investigated to determine the controlling moments are shown in bold type.
6. **Determine the Load Factors** [1.3.2.1] [3.4.1]

$$Q = \eta\Sigma\gamma_i q_i$$

where
Q = factored load
η = load modifier
γ = load factor
q = unfactored loads

a. *Load modifier*: [1.3.3] [1.3.4] [1.3.5]:

$$\eta = \eta_D \eta_R \eta_i > 0.95$$

where
η_D = 0.95
η_R = 0.95
η_i = 0.95

Therefore, η is the maximum of η = (0.95)(0.95)(0.95) = 0.86 or η = 0.95 ⇐ controls.

b. *Load factor*: [Table 3.4.1-1] [Table 3.4.1-2] [3.4.1] [3.3.2]:

γ_{DL} :

Maximum Load Factor	Minimum Load Factor	
$\gamma_{DC\max} = 1.25$	$\gamma_{DC\min} = 0.90$	⇐ slab and barrier rail
$\gamma_{DW\max} = 1.50$	$\gamma_{DW\min} = 0.65$	⇐ future wearing surface

γ_{LL} (Strength-1 Load Combination)

$$\gamma_{LL} = 1.75$$

$$\gamma_{IM} = 1.75$$

TABLE 24.2 Unfactored Moments

Location		Dead-Load Moment (kN–m)			Live-Load Moment (kN–m)			
		M_{DD}	M_{DB}	M_{DW}	$M_{+ve\ LL}$ Alt 1	$M_{+ve\ LL}$ Alt 2	$M_{+ve\ LL}$ Alt3	M_{-veLL} Alt 1
Span 1	Left	0	0	0	0	0	0	0
	.1 pt	–.02	0	0	0	0	0	0
	.2 pt	–.08	0	0	0	0	0	0
	.3 pt	–.18	–.38	0	0	0	0	0
	.4 pt	–.32	–.86	0	0	0	0	0
	.5 pt	–.49	–1.33	0	0	0	0	0
	.6 pt	–.71	–1.81	0	0	0	0	0
	.7 pt	–.97	–2.29	–.01	0	0	0	0
	.8 pt	–1.26	–2.76	–.03	0	0	0	0
	.9 pt	–1.60	–3.24	–.07	0	0	0	0
	Right	–1.98	–3.71	–.11	0	0	0	0
Span 2	Left	–1.98	–3.71	–.11	0	0	0	0
	.1 pt	–1.13	–3.24	.15	3.62	2.64	3.75	2.22
	.2 pt	–.46	–2.77	.34	7.24	5.29	7.51	4.43
	.3 pt	.04	–2.31	.46	10.87	7.93	11.26	6.65
	.4 pt	**.37**	**–1.84**	**.50**	**14.49**	**10.57**	**15.01**	**8.87**
	.5 pt	**.52**	**–1.37**	**.48**	**9.80**	**13.22**	**10.45**	**11.08**
	.6 pt	.50	–.90	.38	5.11	7.75	5.89	9.95
	.7 pt	.31	–.43	.21	.42	2.08	1.33	3.60
	.8 pt	–.05	.04	–.03	–4.28	–3.59	–3.23	–2.76
	.9 pt	–.59	.51	–.34	–8.97	–9.26	–7.79	–9.12
	Right	–1.30	.98	–.72	–13.66	–14.93	–12.35	–15.48
Span 3	Left	**–1.30**	**.98**	**–.72**	**–13.66**	**–14.93**	**–12.35**	**–15.48**
	.1 pt	–.54	.86	–.39	–6.50	–8.61	–5.84	–9.52
	.2 pt	.05	.74	–.12	.67	–2.29	.67	–3.56
	.3 pt	.46	.63	.07	6.00	4.03	5.35	2.40
	.4 pt	.71	.51	.20	4.85	8.72	3.54	8.36
	.5 pt	.78	.39	.25	3.70	6.73	1.74	9.09
	.6 pt	.67	.27	.23	2.55	4.73	–.07	6.47
	.7 pt	.40	.16	.13	1.40	2.74	–1.87	3.86
	.8 pt	–.05	.04	–.03	.25	.74	–3.68	1.24
	.9 pt	–.67	–.08	–.26	–.91	–1.25	–5.48	–1.38
	Right	–1.47	–.20	–.57	–2.06	–3.25	–7.29	–3.99
Span 4	Left	–1.47	–.20	–.57	–2.06	–3.25	–7.29	–3.99

M_{DD}, M_{DB}, M_{DW} represent the moments due to dead loads: deck(including slab and overhang), barrier rail, and future wearing surface moments respectively.

M_{+veLL} and M_{-veLL} represent the positive and negatiave live-load moments, respectively.

c. *Multiple presence factor* [Table 3.6.1.1.2-1]:

$$m_{1lane} = 1.20;\ m_{2lanes} = 1.00;\ m_{3lanes} = 0.85$$

d. *Dynamic load allowance* [3.6.1.2.2] [3.6.2]

$$IM = 0.33$$

7. Calculate the Factored Moments

$$M_u = \eta[\gamma_{DC}(M_{DD})+\gamma_{DC}(M_{DB})+\gamma_{DW}(M_{DW})+(m)(1+IM)(\gamma_{LL})(M_{LL})]$$

TABLE 24.3 AASHTO LRFD Bridge Design Specifications

Type of Deck	Direction of Primary Strip Relative to Traffic	Width of Primary Strip (mm)
Concrete		
• Cast-in-place	Overhang	$1140 + 0.833X$
	Either Parallel or Perpendicular	$+M$: $660 + 0.55S$
		$-M$: $1220 + 0.25S$
• Cast-in-place with stay-in-place concrete formwork	Either Parallel or Perpendicular	$+M$: $660 + 0.55S$
		$-M$: $1220 + 0.25S$
• Precast, post-tensioned	Either Parallel or Perpendicular	$+M$: $660 + 0.55S$
		$-M$: $1220 + 0.25S$
Steel		
• Open grid	Main bars	$0.007P + 4.0S_b$
• Filled or partially filled grid	Main bars	Article 4.6.2.1.8 applies
• Unfilled, composite grids	Main bars	Article 9.8.2.4 applies
Wood		
• Prefabricated glulam		
• Non-interconnected	Parallel	$2.0h + 760$
	Perpendicular	$2.0h + 1020$
• Interconnected	Parallel	$2280 + 0.07L$
	Perpendicular	$4.0h + 760$
• Stress-laminated	Parallel	$0.066S + 2740$
	Perpendicular	$0.84S + 610$
• Spike-laminated	Parallel	$2.0h + 760$
• Continuous decks or interconnected panels	Perpendicular	$2.0h + 1020$
	Parallel	$2.0h + 760$
• Noninterconnected panels	Perpendicular	$2.0h + 1020$
• Planks		Plank width

Note: 1996 Interim Revisions, Table 4.6.2.1.3-1 — Equivalent Strips.

Positive Moment:

a. M_{+ve} *Alternative 1 with one lane of live load* (*0.4 point of span 2*):

$$M_u = 0.95[(1.25)(0.37) + (0.90)(-1.84) + (1.50)(0.50) + (1.20)(1.33)(1.75)(14.49)]$$
$$= 38.03 \text{ kNm} \Leftarrow \text{controls positive moment}$$

b. M_{+ve} *Alternative 2 with one lane of live load* (*0.5 point of span 2*):

$$M_u = 0.95[(1.25)(0.52) + (0.90)(-1.37) + (1.50)(0.48) + (1.20)(1.33)(1.75)(13.22)]$$
$$= 35.20 \text{ kNm}$$

c. M_{+ve} *Alternative 3 with two lanes of live load* (*0.4 point of span 2*):

$$M_u = 0.95[(1.25)(0.37) + (0.90)(-1.84) + (1.50)(0.50) + (1.00)(1.33)(1.75)(15.01)]$$
$$= 32.77 \text{ kNm}$$

Negative Moment:

M_{-ve} *Alternative 1 with one lane of live load* (*0.0 point of span 3*):

$$M_u = 0.95[(1.25)(-1.30) + (0.90)(0.98) + (1.50)(-0.72) + (1.20)(1.33)(1.75)(-15.48)]$$
$$= -42.81 \text{ kNm} \Leftarrow \text{controls negative moment}$$

As specified in [4.6.2.1.1] the entire width of the deck shall be designed for these maximum moments.

In reviewing the magnitude of the dead loads in comparison to the live loads it becomes apparent that the total combined dead load plus live-load moment is clearly dominated by the live-load moments. Therefore, performing complex analysis to determine exact dead-load moments is not justified. Using elementary beam formulas or other approximate methods is probably sufficient in most cases.

8. Determine the Slab Reinforcement Detailing Requirements

a. *Determine the top deck reinforcement cover* [Table 5.12.3-1]:
The top deck requires a minimum cover of 50 mm over the top mat reinforcement, unless environmental conditions at the site require additional cover. This cover does not include additional concrete placed on the deck for sacrificial purposes, grooving, or grinding. The clearance between the bottom mat reinforcement and the bottom of the deck slab is 25 mm, up to a No. 35 bar.

b. *Determine deck reinforcement spacing requirements* [5.10.3.2]:

$$s_{\max} = 1.5(175 \text{ mm}) = 262 \text{ mm} \Leftarrow \text{controls}$$

or

$$s_{\max} = 450 \text{ mm}$$

The minimum spacing of reinforcement is determined by [5.10.3.1] and is dependent on the bar size chosen and aggregate size.

c. *Determine distribution reinforcement requirements* [9.7.2.3] [9.7.3.2]:
Reinforcement is needed in the bottom of the slab in the direction of the girders in order to distribute the deck loads to the primary deck slab reinforcement, which is oriented transversely to traffic. The effective span length (S) is dependent on the girder type, which was not specified for this example in order to make the solution general. However, with the girder spacing of 2050 mm used in this example, the maximum value of 67% in the formula $(3840)/(\sqrt{S}) \leq 67\%$ would control. This value is a percentage of the primary slab reinforcement that is to be used for distribution reinforcement in the bottom of the slab and is placed parallel to the main girders.

d. *Determine the minimum top slab reinforcement parallel to the girders* [5.10.8.2]:
The top slab reinforcement shall be a minimum as required for shrinkage and temperature of $(0.75)(A_g)/f_y$. The top slab reinforcement may be controlled by the negative moment reinforcement needs of the main girders which would likely be greater than the shrinkage and temperature reinforcement requirements.

9. Design

The entire width of the deck should be designed for the maximum positive and negative moments as specified in [4.6.2.1.1]. The positive and negative reinforcement is designed like a typical concrete beam. Concrete design is covered in many civil engineering texts and it is not the intent of this chapter to cover this topic. See Chapter 9 of this text for a discussion of concrete design methods.

24.2.2 Precast Concrete Bridge Decks

This type of bridge deck (Figure 24.3) has the advantage of not requiring significant curing and setup time prior to being loaded with traffic loads as is required for a cast-in-place deck. Therefore, this type of deck is often used for deck replacement [1]. Work can be done overnight or during off-peak traffic times when traffic can be temporarily detoured around the bridge or when a reduced number of traffic lanes can be provided and the deck is replaced in longitudinal sections while traffic continues in adjacent lanes.

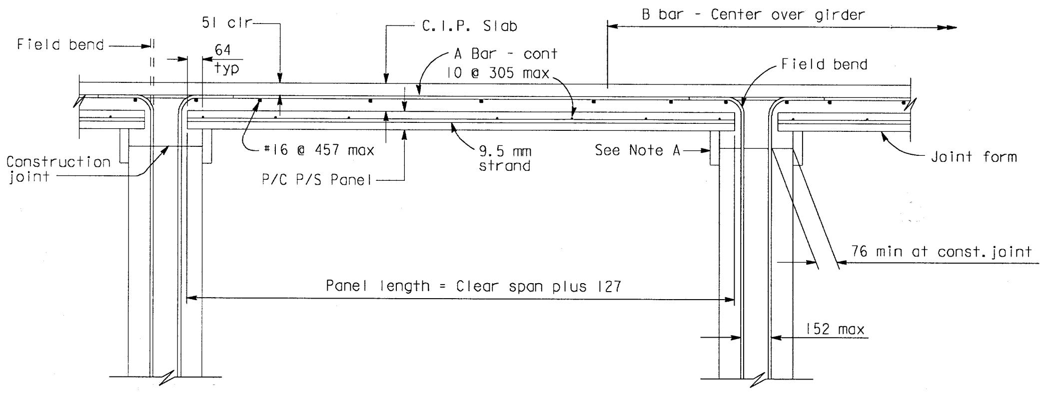

FIGURE 24.3 Precast concrete bridge deck. (From California Department of Transportation, Bridge Standard Detail Sheets, Sacramento, 1993. With permission.)

Precast decks can either serve as the final deck riding surface or a cast-in-place surface can be added on top. A cast-in-place surface uses the precast panels as the deck formwork, which would be placed between the girders. In adding a cast-in-place concrete surface, the problems associated with filling and maintaining the joints between the panels are reduced, and it assists in making the bridge deck composite with the girders. However, this method is at odds with the desire to open the deck to traffic as soon as possible. This led to the development of methods that do not require an additional final surface. If a final concrete surface is not placed on top, the joints between each panel must be successfully filled to avoid leakage and avoid future maintenance problems. This is typically done with expansive grouts or special epoxy crack sealers.

Precast panels may be prestressed, which reduces the depth of the precast panel between the main longitudinal girders or provides for increased spacing between the main girders for a given deck thickness. Perhaps more importantly, prestressing reduces the cracking in the deck. This is especially important for bridges exposed to aggressive environments.

Future widenings of decks using transverse deck prestressing is more difficult than a deck with conventional reinforcement. While a prestressed deck is likely to require less maintenance than a conventionally reinforced deck, repairs that may be required will be more difficult than for a conventionally reinforced deck [1].

24.2.3 Steel Grid Bridge Decks

Steel grids (Figure 24.4) can either be constructed off site as individual panels, constructed at the job site, or can even be assembled on the site of individual components. These grids are then usually welded or mechanically fastened to the supporting components. A significant advantage of open grid decks is their light weight. This can be especially important for existing bridges where the girders would require strengthening if extra dead load were added to the structure, and for movable bridges where dead load is minimized in order to limit loads on the mechanical systems.

However, open grids can result in poor riding quality and a loud whine as traffic crosses them. In addition, rainfall and possible spills fall directly through the deck and cannot be captured or controlled. This can lead to corrosion of components below the bridge and in environmental problems below the bridge because spills can fall directly into waterways. In addition to this, careful detailing is required to avoid fatigue problems associated with steel grid systems [1]. Interlocking grid systems that do not require welding may eliminate stresses that cause these types of failures. Older open steel grid systems have had problems with skid resistance, but newer systems are available that meet today's standards. However, open grids can wear over time, reducing the skid resistance.

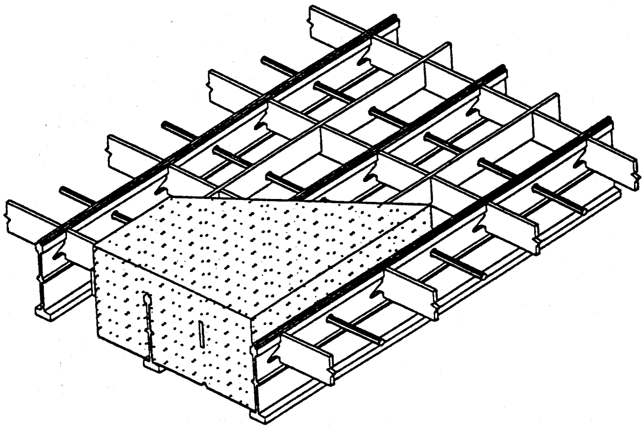

FIGURE 24.4 Steel grid bridge deck. (From American Grid, *Weldless Bridge Deck Systems*. With permission.)

Some of these problems can be solved by filling, or partially filling, the grid with concrete. The grid is then made composite with the concrete and acts as rebar in tension would in a conventionally reinforced-concrete beam. Although this type of deck is heavier than an open grid, it is still lighter than a conventional concrete bridge deck. Typically, an overfill of the grid is specified, which is assumed also to act compositely with the grid. This overfill provides added cover to minimize cracking, reduce corrosion of the steel grid, and improve ridability.

A variation of the filled and unfilled grid type uses an unfilled grid with shear studs. This makes it composite with a reinforced concrete slab which is set on top of the grid in an attempt to combine the advantages of a concrete and a steel grid deck.

24.2.4 Timber Bridge Decks

This was a common deck type prior to the advent of the automobile (Figure 24.5). Because of durability problems this type of deck is rarely used today, except on very low volume bridges, often in rural areas [1]. Timber bridge decks may be constructed using glulam timber panel decks and stress-laminated decks that are post-tensioned together. An asphalt wearing surface may be placed on top of the deck in an attempt to increase the durability of this type of bridge deck.

24.2.5 Steel Orthotropic Bridge Decks

An orthotropic steel deck is a deck plate acting as the flange section and is stiffened with longitudinal ribs and transverse floor beams. A wearing surface is added to act compositely with the deck plate. This subject is covered in greater depth in Chapter 14 of this text.

24.3 Approach Slabs

The structure approach slab provides motorists a smooth transition between the paved highway surface and the riding surface of a bridge. The most common construction consists of reinforced portland cement concrete (PCC). The need for an approach slab is generated by the differential vertical displacement that often occurs between the generally unyielding bridge structure and adjacent fill approaches.

The settlement of the adjacent fill can be gradual, over lengthy periods of time, or sudden, such as when ground motion from an earthquake "liquefies" unconsolidated material in the presence of groundwater. During such a catastrophic event, the role of the approach slab becomes paramount in enabling the passage of emergency vehicles immediately after the earthquake.

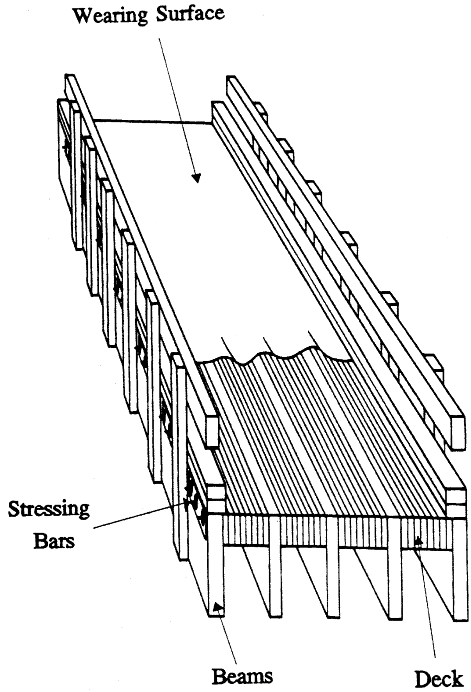

FIGURE 24.5 Timber bridge deck. (From Davalos, J. F., Wolcott, M. P., Dickson, B., and Brokaw J., *Quality Assurance and Inspection Manual for Timber Bridges,* 1992. With permission.)

The design and maintenance of an approach slab is greatly dependent on numerous factors which can affect the amount and rate of settlement that occurs. Careful attention to address these problematic features properly will lead to a serviceable, low-maintenance structure.

24.3.1 Structural Design Considerations

The reinforced-concrete approach slab is designed as a simply supported element, spanning from where it rests on the bridge abutment to the end that meets the roadway. The length of the slab is generally standardized to provide for the majority of applications. Currently, in California, the most commonly used approach slab is one which provides a 9-m transition from roadway to bridge.

The slab itself is usually about 300 mm thick and is reinforced with two-way mats of reinforcing steel both top and bottom. Although the slab is designed to support any passing live load adequately, equal importance is given to preventing the slab from cracking and "breaking up" under the constant cycles of loading by traffic, causing costly maintenance repairs and poor ridability for the motoring public. Reinforcement ratio indexes for the longitudinal bottom reinforcing bars of 0.0110, 0.0022 for the bottom transverse bars, and 0.0031 for the longitudinal top reinforcing bars have historically provided satisfactory performance for the 9-m-long slab. A minimum amount of transverse top reinforcing steel satisfies temperature and shrinkage requirements.

24.3.2 Settlement Problems

Settlement of the adjacent fill abutting a bridge is related to the geologic properties of the fill material and the native soil that underlies it. Considerations should be given to include composition and compaction of the fill material, settlement of the native soil due to the overburden imposed on it, and drainage conditions of the approaches.

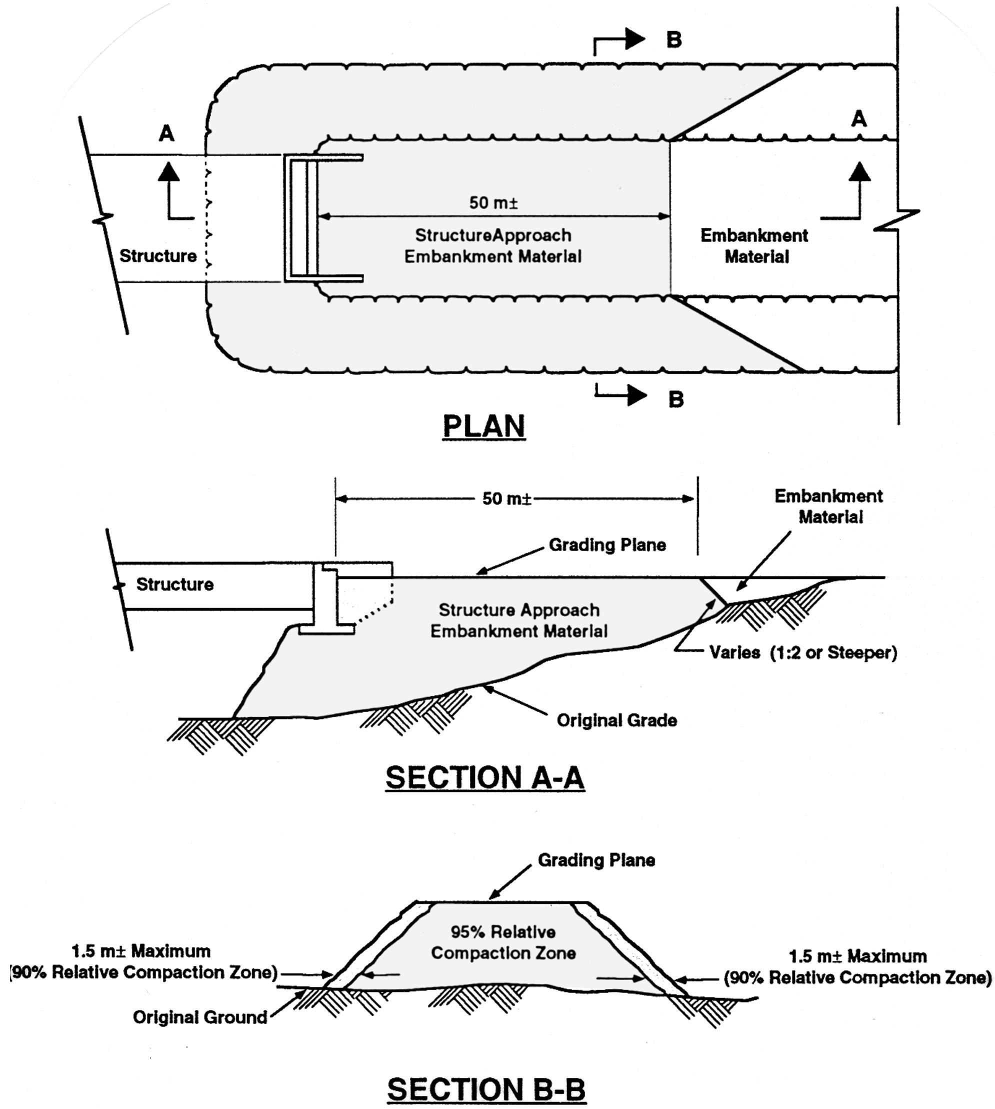

FIGURE 24.6 Limits of structure approach embankment material. (From California Department of Transportation, *Highway Design Manual*, Sacramento, 1995. With permission.)

The selection of fill material that creates the approaches to a structure is critical. Volume changes of the fill material result from rearrangement of soil particles, loss of moisture, gain of moisture, or frost and ice action. The fill material should not consist of soil types that are subject to influence under those conditions, such as cohesionless soils, which tend to settle under the vibration of traffic, or highly plastic soils, which are difficult to work with and highly compressible [3]. Approach fills should be constructed with selected material of slightly cohesive granular soil and extend well beyond the limits of abutments.

Adequate compaction of the fill material will limit the amount of future settlement by removing the potential volume changes of the soil. A minimum relative compaction value of 95% provides a reasonable lower limit that will minimize detrimental magnitudes of settlement of the fill. In the state of California, current practice is to require this type of "structure approach embankment material" 50 m from the back of the abutment (Figure 24.6).

Special attention is required in the areas immediately adjacent to the abutment or wingwalls, where confined spaces may limit the accessibility of large compaction equipment. Typically, small, hand-operated pieces of equipment are resorted to in order to compact the fill in these locations. It is imperative that proper procedures, which should be carefully addressed in the specifications, are adhered to and diligently monitored for quality assurance. This will limit the potential for differential settlement between the fill adjacent to the bridge structure, which is difficult to compact, and the fill away from the structure, which can be compacted by large equipment.

Subsidence of the underlying native soil is another major contribution to the poor performance of a bridge approach. With the additional mass of the overburden in the form of the approach fill, the native soil may not have sufficient strength to support the additional weight and will settle. Several options are possible to alleviate this problem.

The use of a surcharge to preload the approach site and preconsolidate compressible native material can be an effective solution to limit future, postconstruction settlement. The effectiveness of utilizing a surcharge is dependent on the amount of time available for this operation. Naturally, the more time that can elapse before construction of the approaches, the better. Care must be exercised that the amount and rate of applied surcharge does not exceed the shear strength of the native soil. This typically results in the necessity to load the site incrementally [3].

The use of drains in soft and compressible, slow-draining soil can accelerate its consolidation by basically dewatering the site and allowing primary consolidation to occur. This works well in thick, homogeneous layers of clay. Examples of vertical drains are sand drains and wick drains, which allow the migration of water within the soil to the surface where it can be collected and removed.

At sites where relatively shallow thicknesses of unsuitable, compressible soil may exist, a simple solution is to remove this layer of material. The replacement material would then be imported rock or suitable, well-compacted material.

A final alternative may be the use of lightweight fill material to limit the overburden on the native soil. Appropriate materials suitable for this purpose can consist of furnace slag, expanded shale, coal waste refuse, lightweight or cellular concrete, polystyrene foam, and other materials having small unit weights [3].

Proper drainage of the approach site, particularly behind and between the bridge abutment and wingwalls is essential to minimize surface and subsurface erosion and to reduce lateral hydrostatic pressure against these structural elements. An effective drainage system within the approach fill will consist of a medium located immediately adjacent to the abutment and wingwalls that will allow for the migration of water to bottom drains that will move the water away from the area. This can consist of pervious material such as gravel or a geocomposite drain and filter fabric, which is a man-made waffle-like material that channels water vertically downward. The placement of pervious material concurrently with the approach backfill is a difficult and more tedious process than the use of the geocomposite drain. The geocomposite drain system is the preferred method and is applied as shown in Figure 24.7.

24.3.3 Additional Considerations

Some related considerations that appear to have no effect on the performance and serviceability of approach slabs are worth mentioning. These include location of approaches relative to the structure, fill height, and average daily traffic (ADT).

The location of the approach, either leading to or away from the structure, seems to make negligible difference in the performance of the approach slab. One might think that the approach leading away from the structure would experience greater impact from vehicles as they come off the structure, resulting in greater settlement and damage to the approach slab. Studies show, however, essentially no difference between slabs at either end of the structures [5].

The height of approach fill also appears to have little effect on the long-term approach performance. It would seem reasonable to conclude that taller fill heights would exhibit the highest continued

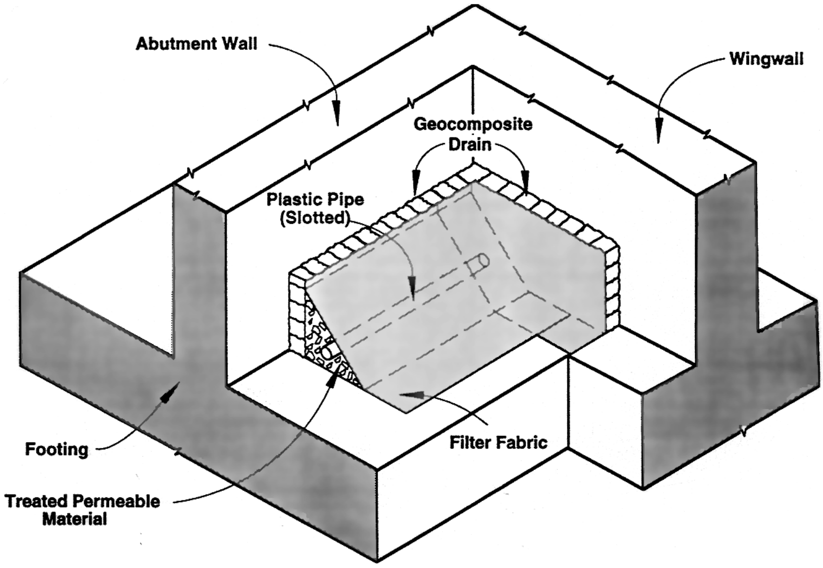

FIGURE 24.7 Abutment drainage details. (From California Department of Transportation, Highway Design Manual, Sacramento, 1995. With permission.)

settlement, but studies again show essentially uniform ground subsidence over all fill heights. A possible explanation is that higher fills are more likely to have been preconditioned in the form of allowing for a settlement period or surcharge of the fill prior to construction of the approach slab [5].

Finally, the volume of average daily traffic similarly exhibits negligible impact to the PCC structure approaches. AC approach data were inconclusive. It appears that the number of cycles of loading has no bearing on the long-term condition of the approach slabs. Maintenance records indicate essentially equal amounts of patching required for varying ADT levels [5].

24.4 Summary

There are several different types of bridge decks available to the design engineer. It is the responsibility of the engineer to determine the most appropriate type of bridge deck for a given site and situation. This can range from a new, short-span bridge with limited traffic to rehabilitation of a long-span bridge over water to be replaced at night because of significant traffic demands.

For most new bridges, cast-in-place concrete bridge decks are chosen as the most appropriate deck type. Typically, these types of decks are designed as a transverse beam supported by the main longitudinal girders. However, just as alternative types of bridge decks are available and are gaining greater acceptance, a new empirical method of designing cast-in-place concrete bridge decks that considers the arching effect between the girders has been introduced. While cast-in-place concrete decks designed as transverse beams have been the standard for decades, bridge deck type and design is continuing to evolve.

The major considerations for approach slabs are settlement of the approach fill, settlement of the underlying native soil, adequate drainage of the fill area behind the abutments and between the wingwalls, and adequate reinforcement; all must be addressed when designing a structure approach slab. Careful selection of suitable fill material and proper consolidation of the fill will limit post-construction settlement. Settlement of the native underlying soil can be eliminated through the practice of applying a surcharge, the use of preconstruction drains in clay soils, the removal of unsuitable compressible soil layers, and, finally, the use of lightweight fills. Proper drainage adjacent to the structure abutments and under the approach slabs will prevent subsurface erosion which would eventually undermine the underlying subbase of the approach slab.

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