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Demetrios E. Tonias
President
HMC Group Ltd.
Saratoga Springs, New York

HIGHWAY ENGINEERING

For the purposes of this section, a highway is considered a conduit that carries vehicular traffic from one location to another. Highway engineering deals with provisions for meeting public needs for highways; environmental impact of highways; planning, design, construction, maintenance, and rehabilitation of highways; access to and exit from highways; economics and financing of highway construction; traffic control; and safety of those using or affected by the use of highways.

Highway engineering is continually evolving. While many of the design techniques employed are the same today as they were fifty or more years ago, new concepts such as Intelligent Vehicle Highway Systems (IVHS) and Pavement Management Systems (PMS) have significant effects on highway engineering. This section presents these newer techniques as well as fundamental principles of highway engineering and their practical applications based on long-time experience.

16.1 Classes of Highways

Highways can range in character from a dirt road in a rural setting to a multilane pavement in an urban environment. They are classified in accordance with functional characteristics. These characteristics are based on the location of the road, such as urban or rural; width of the road, such as single lane or multilane; and the type of service the road is to provide, such as local access or travel between cities.

Principal guidelines for classifying highways are given in the American Association of State Highway and Transportation Officials (AASHTO) guide, "A Policy on Geometric Design of Highways and Streets" (Policy). Highways are grouped in accordance with the type of service they provide; that is, the type of travel associated with the road. Travel is facilitated by a highway **network** that is comprised of various classes of highways. Figure 16.1 presents a schematic of a highway network composed of the three principal highway classes: arterials, collectors, and local roads.

Arterials are highways that provide direct service to major population centers. **Collectors** provide direct service to towns and link up with arterials. **Local roads** connect various regions of a municipality and tie into the system of collectors. Further subdivisions of these three principal categories can be made by defining principal and minor arterials and major and minor collectors.

A principal arterial provides for main movement whereas a minor arterial acts as a distributor. Major and minor collectors are subclassifications

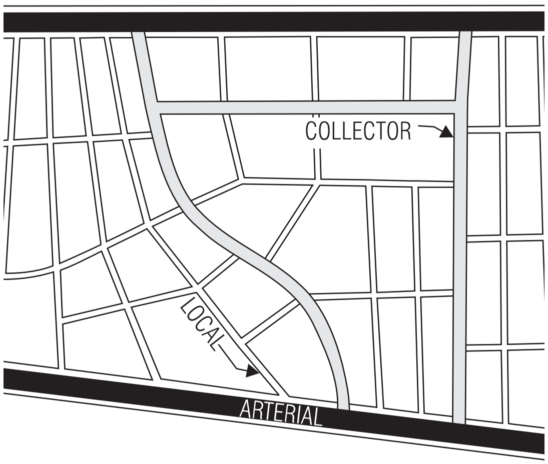

Fig. 16.1 Schematic of a suburban network with local, collector, and arterial roads.

that can be used to define the types of population centers the collector serves and other impacting criteria such as spacing and population density.

Each functional class of highway is designed to meet specific needs. For instance, arterials are built to facilitate a high degree of mobility. This need for mobility typically is met by construction of multilane highways with strict control of access. Local roads, in contrast, are designed to facilitate access to various areas of a municipality; for example, commercial and residential areas.

Also associated with these basic functional classifications are the political classifications of highways. Criteria to which a roadway is constructed and maintained are related to the political entity, such as Federal, state, and county (local) government, that has jurisdiction over the highway. These criteria have a profound impact on the way a highway is designed. A local road in a rural setting, for example, may consist of asphalt and aggregate surfaces on gravel bases and would be financed by local taxes and some state funds. A toll road in the Interstate system, however, would have a higher-quality, durable pavement and would be funded by those who use the highway and by some Federal assistance. Thus design of a highway is highly dependent on whether it will serve a rural or urban environment.

AASHTO defines an urban area as "those places within boundaries set by the responsible State and local officials having a population of 5,000 or more." Furthermore, the AASHTO Policy defines an *urbanized area* as one with a population of 50,000 and over and a *small urban area* as one with a population between 5000 and 50,000. *Rural areas* are defined as areas falling outside the definition of urban areas.

16.1.1 Rural Highway Systems

A **rural principal highway system** is comprised of those highways that offer corridor movement and are capable of supporting statewide or interstate travel on this class of highway. Rural principal arterial systems can be further subdivided into freeways and all other principal arterials.

A **freeway** is a divided highway with fully controlled access. Access to a freeway is made *without* use of at-grade intersections. Figure 16.2 shows a schematic of a rural highway network and its corresponding functional classifications. As illustrated in this figure, the arterials provide direct service between cities and large towns, which are the major traffic generators.

Rural minor arterial systems serve in conjunction with principal arterial systems to link together cities, large towns, and other traffic generators, for example, large resorts. The resulting network can also serve to integrate interstate and intercounty service. To provide for a consistent and high degree of mobility, maximum travel speeds are set as high as those on the associated principal arterial systems and therefore require a design that can accommodate such speeds.

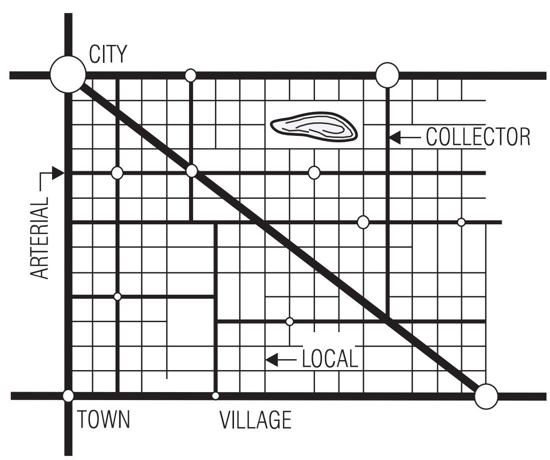

Fig. 16.2 Schematic of a rural highway network serving towns, villages, and cities.

Rural major collector systems contain routes that are intended to serve county seats and nearby large towns and cities that are not directly served by an arterial system. Other traffic generators that may be served by a major collector system are consolidated schools, shipping points, county parks, agricultural areas, and other locations of intracounty importance. In Fig. 16.2, the collectors are shown to collect traffic from local roads, which service specific land uses such as farms, and distribute this traffic to the arterials.

Rural minor collector systems contain routes that carry traffic from the system of local roads and other traffic generators of local importance to other facilities.

Rural local road systems include all roads in the rural system that do not fall into any of the preceding rural categories. These local roads carry traffic from land adjacent to the collector system and are useful for travel over relatively short distances.

16.1.2 Urban Highway Systems

An urban principal arterial system is designed to accommodate travel along heavily traveled corridors serving major centers of activity in urban areas. Although an urban principal arterial system may or may not be a controlled-access facility, all controlled-access facilities are classified as this type of system.

An urban principal arterial system is integrated with its rural counterpart that serves major centers. The urban system facilitates most of the trips that either enter or leave a population center in addition to serving most through traffic.

There are three major subclassifications of an urban principal arterial: interstate, other freeways, and other principal arterials. The last type may provide partial or no controlled access. Only this subclassification of primary arterial can be used to provide direct access to intersecting roads.

Urban minor arterial systems interconnect and augment an urban principal arterial system. In comparison to the primary arterials, a minor arterial system is intended more for use for direct access to intersecting roads and less for provision of travel mobility. While such arterials do not usually pass through identifiable neighborhoods, they may support local bus routes and provide some continuity between various communities in an urban area.

Urban collector street systems receive traffic from local roads in commercial, industrial, and residential areas for travel to an arterial system. Collector street systems may carry local bus routes and, in some instances, comprise the entire street grid of a central business district.

Urban local street systems include all roads in the urban system that do not fall into any of the preceding urban categories. These roads carry traffic from land adjacent to the collector system. Through traffic is generally discouraged.

Elements of Highway Transverse Cross Sections

The geometry of a typical highway comprises three basic components: cross-sectional geometry, horizontal geometry, and vertical geometry. The type, size, and number of elements used in a highway are directly related to its class (Sec. 16.1) and the corresponding function of the highway.

16.2 Travel Lanes

Travel lanes are that section of a roadway on which traffic moves. Figure 16.3 shows a typical two-lane highway and such cross-sectional components as travel lanes, shoulders, and side slopes.

From a geometric standpoint, the key parameters defining a travel lane are the number of lanes, their width, and cross slopes, all of which impact the level of service a highway can accommodate. Of equal importance are the characteristics of the pavement surface and its skid resistance. These features affect the overall ridability, safety, and future maintenance of a highway.

Travel-Lane Widths ■ Travel lanes generally range in width from 10 to 13 ft. (Under extreme circumstances, a width of 9 ft may be used.) The width selected has a significant impact on highway capacity. A width of 12 ft predominates on high-type pavements since the cost differential for constructing a 12-ft lane width instead of a 10-ft width is usually offset by reduced maintenance costs for the shoulders and the edges of pavements. When the narrower, 10-ft width is used, the shoulders and edges of pavement undergo more wear and tear from wheel concentrations at these locations.

Number of Travel Lanes ■ For most highways, three lanes in one direction usually is the

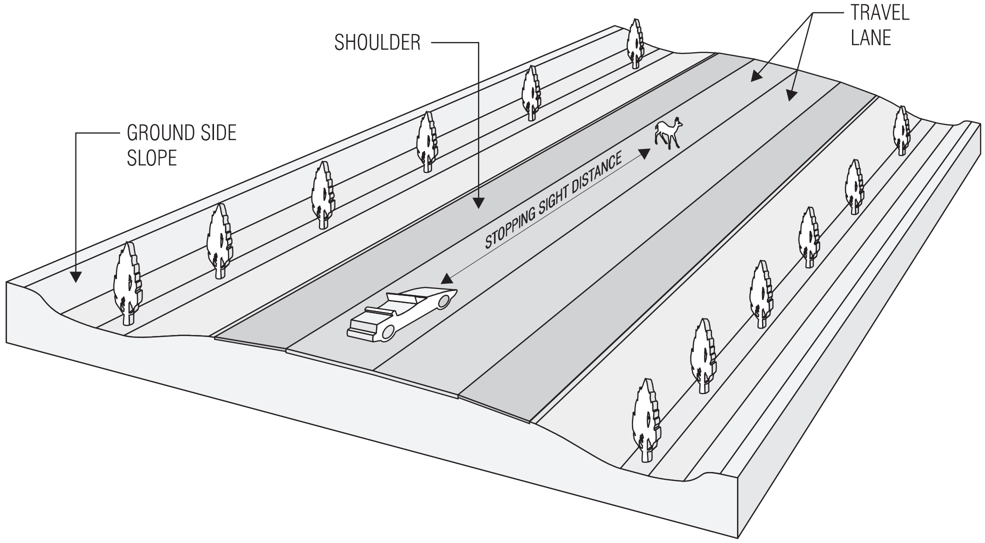

Fig. 16.3 Cross section of a typical two-lane highway.

maximum installed. In certain situations, four lanes in one direction may be provided. If more than three lanes are required, however, and sufficient land is available, dual roadways should be constructed in each direction.

In general, the number of lanes selected should be based on the design traffic volume and other design-related considerations. For example, highways in steep or mountainous terrain may necessitate incorporation of a separate *climbing lane* for slow-moving trucks. Another example is the addition of an exclusive *bus lane* to the lanes that would otherwise be provided. As another example, a *reversible lane* may be used to expedite traffic flow on highways on which traffic flow fluctuates greatly from morning to night (owing to commuting patterns). A general rule of thumb is that lane changes should be avoided at intersections and interchanges.

16.3 Roadway Cross Slopes

For highways with two lanes or more, the roadway usually is sloped from a high point, or crown at the middle of the roadway downward toward the opposing edges. Figure 16.4 shows a typical two-lane highway with linear opposing cross slopes intersecting at the centerline of the total travel width.

Alternatively, the roadway cross slope may be unidirectional. This roadway cross section is generally more pleasing to drivers since vehicles appear to be pulled in the same direction when changing lanes.

Opposing and unidirectional cross slopes have advantages and disadvantages for drainage of the highway. Opposing cross slopes have the advantage of being able to drain the roadway quickly during a heavy rainstorm. This layout, however, requires installation of drainage facilities on both sides of the roadway. Unidirectional cross slopes tend to drain more slowly, but they have the advantage of permitting drainage facilities to be consolidated along one edge of the roadway and thereby reduce construction and maintenance costs.

In some instances, a parabola is used in lieu of straight-line segments in forming the crown of the roadway. While the parabolic section provides good drainage of the roadway, it has relatively high construction costs, and difficulty may be encountered in grading at intersections.

When specifying travel-lane cross slopes, designers should consider the necessity of both adequate drainage and driver safety. A cross slope that is too flat will not drain properly and one that is too steep can cause vehicles to drift toward the edges of pavement, especially when the pavement is slick.

The slope selected generally depends on the type of pavement used. The American Association of State Highway and Transportation Officials (AASHTO) recommends a cross slope of 1.5 to 2% for highways with high-type pavements, 1.5 to 3% for intermediate-type pavements, and 2 to 6% for low-type pavements (Art. 16.4).

From a safety standpoint, cross slopes greater than 2% should be avoided for high-type pavements on which high speeds are permitted and that have opposing cross slopes. Such steep slopes pose a hazard for drivers because, when passing another vehicle on two-lane highways with such pavements, a driver must cross and then recross the crown, where a total cross-slope change (rollover) of more than 4% occurs. In some instances, though, it may be necessary to use slightly steeper cross slopes to facilitate proper drainage. In doing this, however, designers should limit the total cross-slope change to minimize hazards to safe driving.

16.4 Types of Roadway Surfaces

The rate of cross slope specified generally depends on the type of pavement utilized. The American Association of State Highway and Transportation Officials (AASHTO) recognizes three major types of pavement: high-type, intermediate-type, and low-type.

Pavement classified as high-type possesses a wearing surface that can sustain heavy vehicles and high-volume and high-speed traffic over long periods of time without failure due to wear or fatigue. This type of pavement should keep nonroutine maintenance to a minimum and sustain a consistent flow of traffic without repair-related interruptions. Intermediate-type pavements are similar to high-type pavements except that they are constructed in accordance with standards that are not as strict as those for high-type pavements. Low-type pavements, predominately used in low-cost roads, may be composed of surface-treated earth and stabilized materials or any of a variety of loose surfaces, such as earth, shell, crushed stone, or bank-run gravel.

The type of pavement to select depends on a variety of factors of which design speed is only one. For instance, skid resistance is an important factor. See also Arts. 16.18 to 16.24.

Skid Resistance ■ The ability of a pavement to accommodate driver braking and steering maneuvers is a function of the pavement skid resistance, the ability of a pavement to prevent accidents due to skidding. While most pavements perform adequately in dry conditions, their ability to limit skidding may deteriorate under wet or icy conditions. The principal causes of skidding are surface rutting, polishing, bleeding, and lubrication.

When rutting occurs in a pavement, water accumulates in wheel tracks and can cause a vehicle to skid. Polishing can reduce and bleeding of a lubricating substance from the pavement can cover the pavement microtexture, thereby diminishing the harsh surface features that otherwise penetrate thin

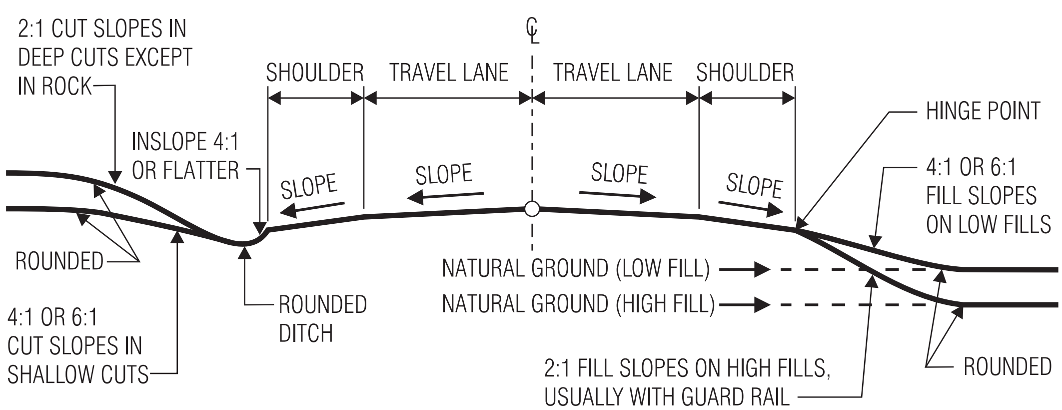

Fig. 16.4 Typical two-lane highway with linear cross slopes.

water film and offer skid resistance. Lubrication of a pavement surface with dust, organic matter, oil, moisture, ice, sand, or other deposits can cause a reduction in or complete loss of skid resistance.

16.5 Shoulders

A shoulder (also known as a **verge**) is that part of a roadway between the edge of the traveled way and the edge of an adjacent curb, ground side slope, or drainage feature, such as a ditch or gutter (Fig. 16.5). A shoulder is designed to accommodate stopping and temporary parking of vehicles, emergency use, and lateral support of base and surface courses. Shoulders should be capable of sustaining starting, stopping, and movement of vehicles without appreciable rutting.

Shoulder Widths. Shoulders usually used range in width from 2 ft for minor local roads to 12 ft for major highways. Figure 16.5 shows various forms of graded and usable shoulder widths. Graded shoulder width is the distance from the edge of the traveled way to the intersection of the shoulder slope and the start of the ground side slope. Usable shoulder width is that part of the shoulder that drivers can use to stop and park a vehicle. If the ground side slope is 4:1 or flatter, then the usable width will be the same as the graded width.

A minimum distance of 2 ft should be maintained between the edge of the traveled way and a vehicle stopped on a shoulder. This results in shoulders with a width of at least 10 ft (preferably 12 ft) for heavily traveled roads. For minor roads, topography or other site-related constraints may necessitate use of smaller shoulder widths. In such conditions, a minimum width of 2 ft sometimes is used, but a range of 6 to 8 ft is preferable.

Usable shoulder widths used on the approach roadways to bridges should also be maintained on the bridge. Narrowing or elimination of the shoulder width on a bridge can create unsafe conditions when emergency stopping of vehicles on a bridge is required.

Shoulders are generally continuous along the length of the roadway. [In some European countries, intermittent shoulders (turnouts) are used on minor roads.] According to AASHTO, some shoulder is better than no shoulder at all. Even under the most severe topographic constraints, designers should endeavor to maximize the width and continuity of shoulders.

Shoulder Cross Slopes ■ The cross slope to be used for a shoulder depends on the type of shoulder construction. Cross slopes ranging from 2 to 6% are generally used for bituminous and concrete surfaces, from 4 to 6% for gravel or crushed rock surfaces, and up to about 8% for turf shoulders. These values are presented as a guide and are neither maximum nor minimum values. It is noteworthy that the highway geometry can greatly impact the design of shoulder cross slopes. For example, long-radius, curved alignments or superelevated roadways can present drainage and other design conditions that require modification of the preceding slopes.

The region in the vicinity of the intersection of the shoulder and ground side slope may be rounded (Fig. 16.5*b* and *c*). When the side slope is 4:1 or

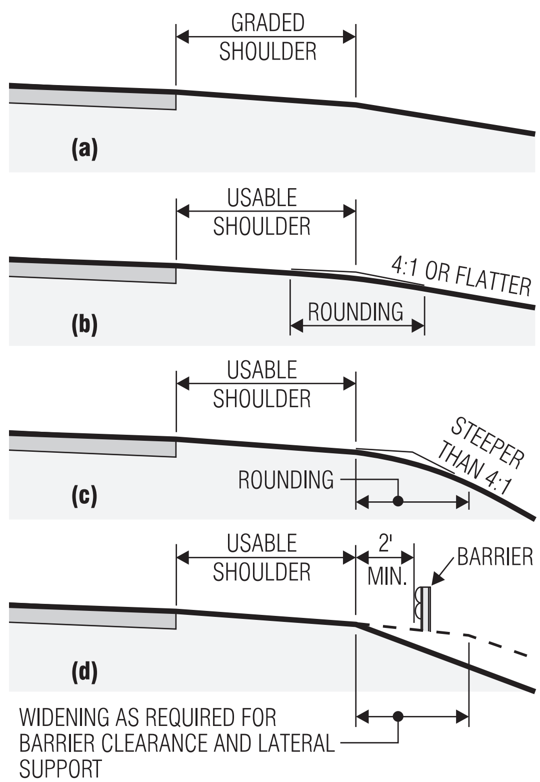

Fig. 16.5 Cross section of a highway with shoulders. (*a*) Graded shoulder. (*b*) Shoulder with side slope of 4:1 or less. (*c*) Shoulder with side slope exceeding 4:1. (*d*) Shoulder wide enough to permit installation of a guard rail, wall, or other barrier. Such vertical elements should be offset at least 2 ft from the outer edge of the usable pavement.

flatter, the rounding may be 4 to 6 ft wide without adverse impact on the usable shoulder width.

If a barrier is installed outside a shoulder, there should be at least 2-ft clearance between the barrier and the usable shoulder, which should be widened as needed for barrier clearance and lateral support (Fig. 16.5*d*). If curbs are placed on the outer side of shoulders, the design should ensure good drainage to prevent excessive ponding. In extreme conditions, ponding can encroach into the traveled way and hinder traffic or cause accidents.

Shoulder Stability ■ Shoulders should be designed not only to support vehicle loading without appreciable rutting but also to be contiguous with the traveled way. They must be constructed flush with the paved surface of the traveled way if they are to function properly. In addition, shoulders should be stabilized so that they remain flush in service.

Shoulders that are not properly stabilized can settle enough to adversely affect a driver's control of a vehicle moving from the traveled way to the shoulder. This situation can also encourage drivers to avoid the pavement edge adjoining the shoulder intentionally, thereby increasing the chance of accidents.

Shoulder-Pavement Contrast ■ It is desirable to vary the color and texture of a shoulder from that of the travel lanes. The resulting contrast serves the dual function of providing clear differentiation between travel lanes and shoulders and discouraging the use of shoulders as through lanes.

Bituminous, gravel, crushed rock, and turf shoulders offer excellent contrast with concrete lanes. For bituminous lanes, one method of enhancing contrast between the travel lanes and shoulders is to seal-coat the shoulders with lighter-color stone chips. A drawback to this method is that contrast may diminish with time. Additional contrast can be provided by installation of reflective striping at the edge of the traveled way.

16.6 Curbs

A curb is a raised element that is used, among other things, to denote the edge of a roadway. Curbs can be made of portland cement or bituminous concrete, granite, or some other hard material. In addition to pavement delineation, curbs provide drainage control, right-of-way reduction, enhanced appearance, delineation of pedestrian walkways, and reduction of maintenance operations. To facilitate drainage, curbs can be combined with a gutter to create a combined curb-gutter section.

There are two general classifications of curbs: barrier and mountable. Figure 16.6 illustrates various types of curbs.

Barrier Curbs ■ The purpose of a barrier curb is to prevent or limit the possibility of a vehicle's leaving the roadway. For this purpose, a barrier curb is made relatively high and given a steep face (Fig. 16.6*a*). Typical height is 6 to 9 in. When the roadway-side face is sloped, the batter should not exceed 1 in horizontal on 3 in vertical.

Barrier curbs are typically used along the faces of long walls and tunnels and along low-speed, low-volume roadways but rarely along major highways. Because of their height, these curbs can present a hazard to vehicles traveling at high speeds, inasmuch as drivers can lose control of their vehicles on contacting the curbs A general rule of thumb is that barrier curbs should not be used when the design speed is greater than 40 mi/h.

Mountable Curbs ■ A mountable curb offers the advantage that a vehicle can cross it when necessary. Typical forms of mountable curbs are illustrated in Fig.16.6*b* to *g*. In contrast to barrier curbs, mountable curbs are relatively low and have flat sloping faces.

To facilitate vehicle crossing of the curbs, the curb faces on the roadway side may be rounded. Curb height depends on the face slope. For face slopes steeper than 1:1, a height of 4 in or less is desirable. For face slopes that fall between 1:1 and 2:1, curb height is limited to a maximum of about 6 in.

Mountable curbs may be installed along median edges to delineate islands. Like barrier curbs, however, mountable curbs should not be used along the travel-way edges of high-speed, high-volume highways. Mountable curbs are often used along the outer edge of a shoulder for drainage control, reduction of erosion, and enhanced delineation.

Color and texture of mountable curbs should be contrasted with that of the adjacent roadway to enhance their visibility, especially at night and in adverse weather conditions. One method used to enhance visibility of curbs is to apply reflective surfaces. Another approach is to form on the curbs depressions and ribs that reflect headlight beams.

16.7 Sidewalks

Sidewalks are used predominately in urban environments, but they are also used in rural areas that are adjacent to schools or other regions, such as shopping centers, where pedestrian traffic is high and sidewalks can help minimize pedestrian-related accidents. Because of their expense, use of sidewalks must be warranted before they are incorporated in a highway cross section. A shoulder can sometimes fulfill the role of a sidewalk if it is constructed and maintained in a way that encourages pedestrian use. Sidewalks, when installed, however, should always be separated from a shoulder, preferably by a curb (Fig. 16.7).

Typical width of sidewalks is 4 to 8 ft. For areas with a large amount of pedestrian traffic, a sidewalk should be at least 6 ft wide.

Sidewalks should be constructed of weather-resistant materials. They should be maintained free from debris and vegetation growth. When

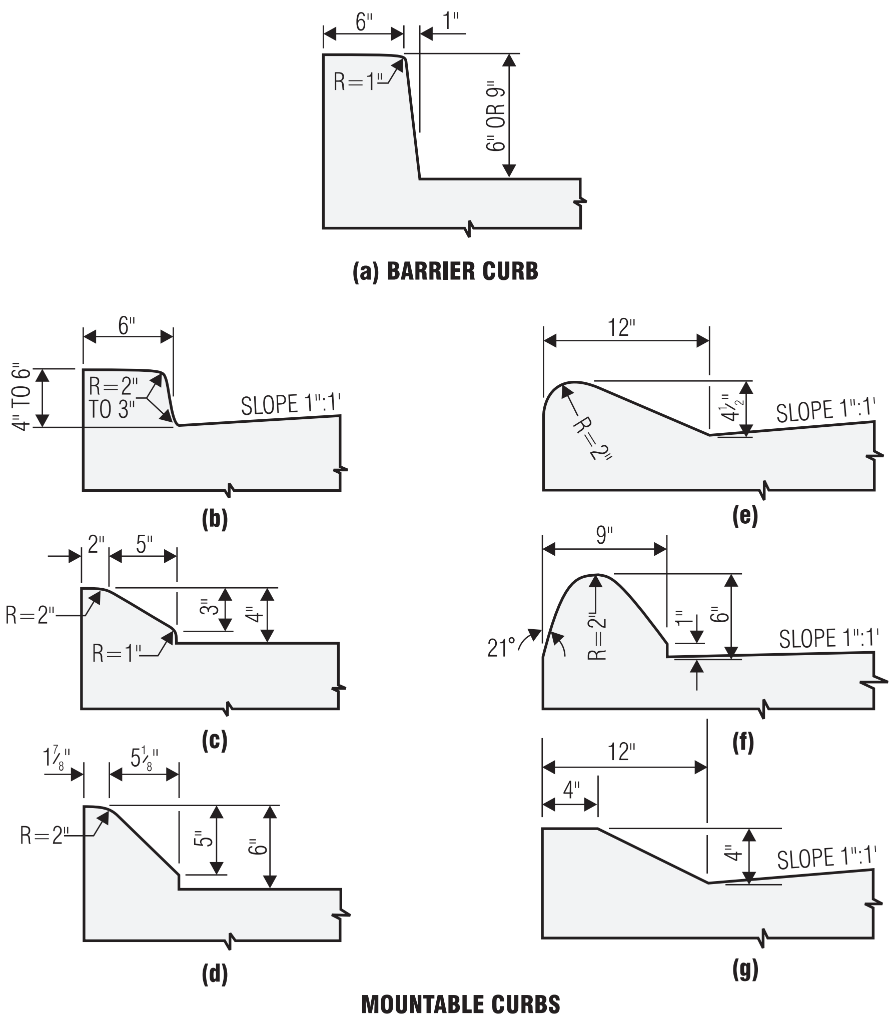

Fig. 16.6 Typical highway curbs. (*a*) Barrier curb used to prevent vehicles from leaving the roadway. (*b*) to (*g*) Mountable curbs that permit vehicles to cross when necessary. Slopes of curb faces and rounding vary.

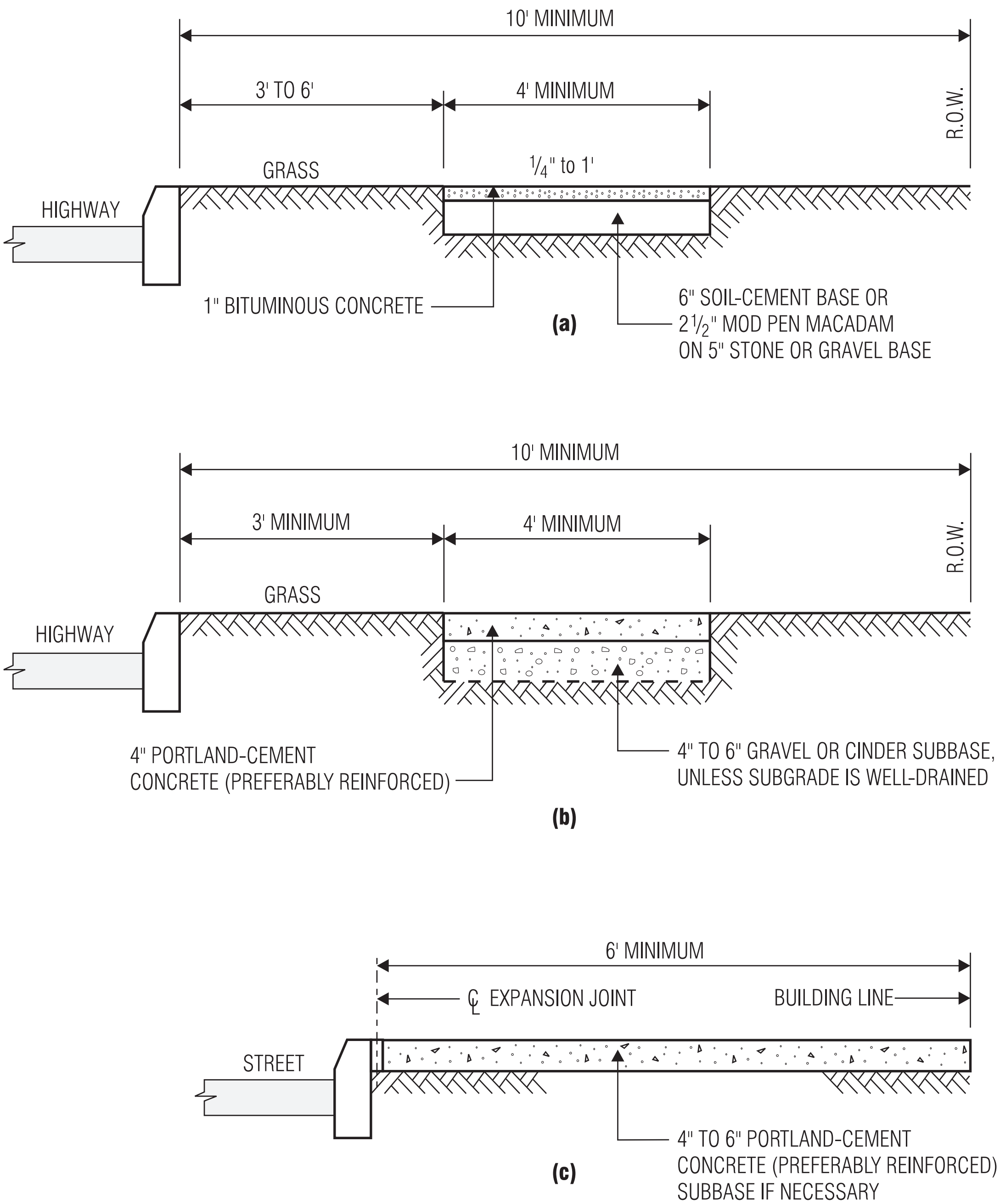

Fig. 16.7 Cross sections of sidewalks: (*a*) for rural or suburban areas; (*b*) for suburban or urban areas; and (*c*) for city streets in a business district.

allowed to deteriorate because of poor maintenance, sidewalks will go unused because pedestrians will choose to walk on the travel lanes rather than the sidewalks. Not only does this defeat the intended function of the sidewalks (and justification for the additional expense) but it also greatly increases the risk of pedestrian-related accidents.

16.8 Traffic Barriers

Roadside barriers are used to protect vehicles and their occupants from impact with natural or man-made features at the side of the road. In addition to protecting vehicles, traffic barriers can also be used to shield pedestrians, construction crews, or cyclists from errant traffic. In its most basic form, a traffic barrier is designed to prevent a vehicle leaving the traveled way from striking a fixed object. The barrier must first contain an errant vehicle and then redirect it. Because of the variable nature of vehicle impacts and the destructive effects at high speeds, extensive full-scale crash tests should be conducted to ensure the adequacy of the traffic barrier to be used.

Barriers are available in a large variety of sizes and shapes. Choice of type of barrier to use depends on a variety of factors, including the environment in which the highway is located and the speed and volume of traffic.

Traffic barriers may be classified as longitudinal barriers, bridge railings and barriers, and crash cushions.

16.8.1 Longitudinal Barriers

Longitudinal barriers can be classified as roadside barriers and median barriers. Whereas a roadside barrier may be placed on either side of a roadway, a median barrier is placed between lanes of traffic traveling in opposite directions.

Barriers differ in the amount of deflection they undergo when struck by a vehicle. The principal categories of longitudinal barriers, based on the amount of deflection allowed, are flexible, semirigid, and rigid systems. Table 16.1 presents some basic forms of roadside barriers as given in the "Roadside Design Guide," American Association of State Highway and Transportation Officials (AASHTO), which discusses selection and implementation of traffic barrier systems.

Flexible systems are designed for large deflections on impact. The primary objective is to contain rather than redirect an impacting vehicle. A flexible barrier generally consists of a weakly supported vertical post and a longitudinal member, such as a cable or a railing, designed to resist most of the tensile impact forces (Fig 16.8*c*). When subjected to an impact, the cable or beams separate from the post, offering little or no resistance in the area of impact.

Semirigid systems utilize the combined strength of the post and the longitudinal member (Fig 16.8*b*). Posts at the point of impact help distribute impact forces to adjacent posts while posts outside the zone of impact help control the deflection of the railing. By limiting deflection, the outside posts assist in redirecting the impacting vehicle along the flow of traffic.

Rigid systems do not deflect appreciably when impacted by a vehicle. Instead, the impact forces are dissipated by raising and lowering the errant vehicle. Energy is also dissipated through deformation of the vehicle's sheet metal. One example of a rigid system is the concrete *Jersey barrier* used in construction zones (Fig. 16.8*a*). Rigid systems are primarily used in sections of highways where the angle of impact will be very shallow since little barrier deflection may occur. They also are used in front of bridge piers that are close to the flow of traffic, because, as a consequence of the limited deflection, they offer a high degree of protection to the hazard object.

While the main body of a longitudinal barrier is a safety device, an exposed end segment of barrier presents a significant hazard to oncoming traffic. Therefore, tapering or burying of the end section, or both, is a necessity. Another option is incorporation of some form of crash cushion or breakaway cable terminal.

16.8.2 Bridge Railings and Barriers

Bridge railings are installed on a highway bridge to prevent vehicular or pedestrian traffic from falling off the structure. They are an integral element of the bridge and thus must be designed to take into account the effects on the bridge superstructure of a vehicle impact.

"AASHTO Standard Specifications for Highway Bridges" presents guidelines for the design of highway bridge railings (See also Art. 17.3.) The type of barriers provided on a bridge depends on the size of the structure, the volume of traffic passing over it, and the type of traffic, such as vehicular only or vehicular with pedestrians.

At each end of a bridge, a transition should be provided between the bridge railings and the approach railings. Since the two railings generally differ in stiffness, a sufficient length of transition railing should be provided to accomplish the change in stiffness smoothly so that snagging or pocketing of an impacting vehicle cannot occur.

16.8.3 Crash Cushions

Also known as an impact attenuator, a crash cushion protects against a head-on collision of an errant

Table 16.1 Standard Sections for Roadside Barriers

Barrier type	Description	Vehicle weight, lb	Maximum deflection, ft
	Flexible		
3-strand cable	$^3/_4$-in-diameter steel cables 3 to 4 in apart, mounted on weak posts spaced 12 to 16 ft	1800–4500	11.5
W-beam weak post	Similar to cable guardrail except it uses a corrugated metal rail whose cross section resembles the letter w	1800–4000	7.3
Thrie beam* weak post	Same as the weak-post, W-beam except it uses a thrie beam rail	1800–4500	6.2
	Semirigid		
Box beam	Consists of a box rail mounted on steel posts (e.g., 6 in × 6-in box mounted on S3×5.7 steel posts on 6-ft centers)	1800–4000	4.8
Blocked-out W-beam (strong post)	Consists of wood or steel posts and a W-beam rail. Posts are set back or *blocked out* to minimize vehicle snagging	1800–4500	2.9
Blocked-out thrie beam* (strong post)	Same as blocked-out W-beam except with a thrie-beam rail. The added corrugation stiffens the system	1800–4000	3.3
Modified thrie beam*	Similar to a blocked-out W-beam with a triangular notch cut from the spacer block web. Minimizes vehicle rollover	Tested for 1800 lb, 20,000 lb (2.9-ft deflection), and 32,000 lb	
Self-restoring barrier (SERB) guardrail	Consists of tubular thrie beam rail supported from wood posts by steel pivot bars and cables. Classified experimental	1800–40,000	3.9
Steel-backed wood rail	Consists of wood rail backed with a steel plate and supported by timber posts	1800–4500	
	Rigid		
Concrete safety shape	Similar to a concrete median barrier but has a smaller section. Has sloped front face and vertical back face	1800–4500	
Stone masonry wall	A 2-ft-high barrier consisting of a reinforced concrete core faced and capped with stone and mortar	1800–4300	

*Cross section of a thrie beam looks like three vees (vvv).

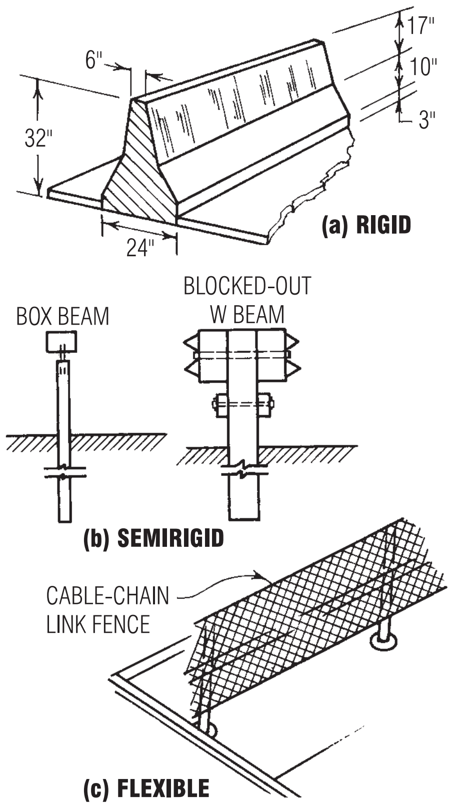

Fig. 16.8 Typical barriers for roadways.

vehicle with a hazard by decelerating the vehicle to a safe stop or redirecting it from the hazard. The goal of crash cushions is to minimize the effects of accidents rather than to prevent them. In essence, a crash cushion limits the effects on a vehicle of a direct impact by absorbing the energy of the impact at a safe, controlled rate.

A crash cushion often is used at a critical location containing a fixed object. One such location is at a ramp gore (triangular area between an exit ramp and a roadway) where the highway and ramp railings join at a sharp angle. Another critical location is at obstacles, such as toll booths, that are installed directly in the flow of traffic.

Crash cushions usually are proprietary systems that are designed and tested by their manufacturers. Most of the systems are based on either absorption of kinetic energy or transfer of momentum to an inertial barrier.

To absorb kinetic energy, plastically deformable materials or hydraulic energy absorbers are placed in the front of the hazard. Dissipation of energy is also achieved through deformation of the front portion of an impacting vehicle. Rigid backup or support is provided to resist the impact force that causes deformation of the crash cushion. The goal of the system is primarily to protect the occupants of the impacting vehicle from injury and secondarily to preserve the integrity of the obstacle.

For transfer of momentum to an inertial barrier, an expendable mass of material is placed in the path of the vehicle to absorb impact. For example, containers filled with sand may be used as an inertial barrier (Fig. 16.9). If a vehicle were to impact such a crash cushion, the sand would absorb momentum from the vehicle. The momentum of the vehicle and the sand after impact would equal the momentum of the vehicle just before the impact occurred. While theoretically the vehicle would not come to a stop, the loss in momentum of the vehicle would be sufficient to slow the vehicle to a speed of about 10 mi/h after impact with the last container. Design of crash cushions is typically accomplished through use of manufacturer-supplied design aids and charts.

16.9 Highway Medians

A median is a wide strip of a highway used to separate traffic traveling in opposite directions (Fig. 16.10). The width of the median in a two-lane highway is the distance between the inner edges of the lanes and includes the shoulders in the median. The width of the median for a highway with two or more lanes in each direction is the distance between the inner edges of the innermost lanes and includes the shoulders in the median.

In addition to separating the opposing flows of traffic, a median is designed to accomplish the following:

- Offer a recovery area for errant vehicles
- Provide an area for emergency stopping
- Serve as a safe waiting area for left-turning and U-turning vehicles
- Decrease the amount of headlight glare
- Allow for expansion to future lanes

Medians may be flush, raised, or depressed. Figure 16.10 shows these basic forms in various configurations. Both flush and raised medians are generally used in urban environments whereas depressed medians are often used in high-speed freeways. Medians should be contrasted in color and texture with the roadways for maximum visibility.

Widths used for medians generally range from 4 to 80 ft. In general, the wider the median, the better. For one thing, median widths of 40 ft or more provide a distinct separation of noise and air pressure from the opposing lanes. For another, incorporation of large green spaces with plantings can create an aesthetically pleasing appearance. Another consideration is that, depending on the width of the median, a traffic barrier may or may not be required. The larger the median width, the less the need for a barrier. Installation of a median barrier should be investigated for narrow medians (those less than 30 ft wide) and for medians that a vehicle out of control may be expected to cross and encounter traffic in the opposite direction. A balance should be struck, however, between the cost of increased median width and the overall cost of the project. In addition to economics, the psychology of drivers is also an important consideration in design of a median.

Design of medians should also take into account the possibility of their use to reduce glare from headlights in the opposing travel lanes. Visibility can be decreased by glare and shadows resulting from oncoming headlights. This condition may be especially acute when raised medians are used. It can be corrected through the incorporation of antiglare treatments in conjunction with a median barrier.

Regardless of the type of median chosen, drainage is an important design consideration. Flush and raised medians should be crowned or depressed for proper drainage. Depressed medians located on freeways should be designed to accommodate drainage and snow removal. For drainage, a ground slope of 6:1 is often used, but a slightly flatter slope may be adequate. Drainage inlets and culverts should be provided as necessary for runoff removal.

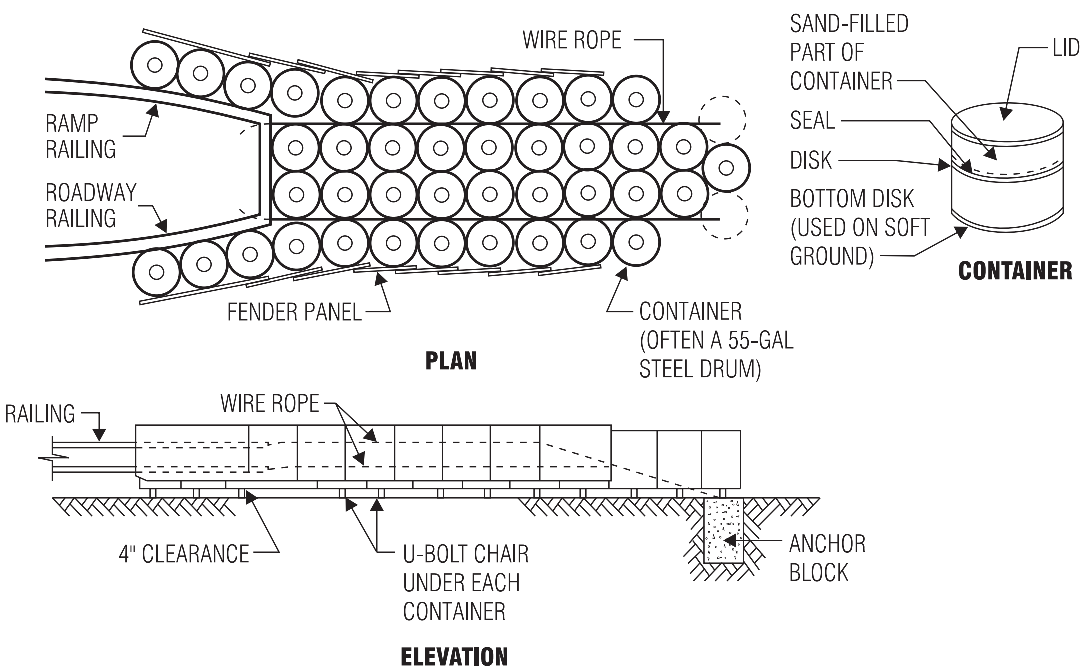

Fig. 16.9 Containers filled with sand used as an inertial barrier.

16.10 Highway Roadside

This is the area that adjoins a highway and can be used to accommodate drainage facilities and for recovery of errant vehicles (Fig. 16. 11). (Shoulders are not included in this area.) A roadside, however, can contain hazards to vehicles that leave the roadway, causing them to come in contact with obstacles or topography they cannot traverse.

A typical roadside that is not flat may contain one or more of the following elements: embankment or fill slope (negative slope), cut slope (positive slope), drainage channel or ditch (change in slope, usually negative to positive), clear zone, curb, sidewalk, berm, fence, traffic barrier, noise barrier, and highway light posts.

16.10.1 Clear Zone

Selection of width, slope, and other characteristics of roadside elements should provide for recovery of errant vehicles. To facilitate design of safe side slopes and related roadside elements, the American Association of State Highway and Transportation Officials (AASHTO) recommends establishment of a clear zone defined as that "area beyond the edge of the traveled way which is used for the recovery of errant vehicles." The traveled way does not include shoulders or auxiliary lanes.

The width to be used for a clear zone depends on traffic volume and speed, and embankment slopes. Rural local roads and collectors that carry low-speed traffic should have a minimum clear-

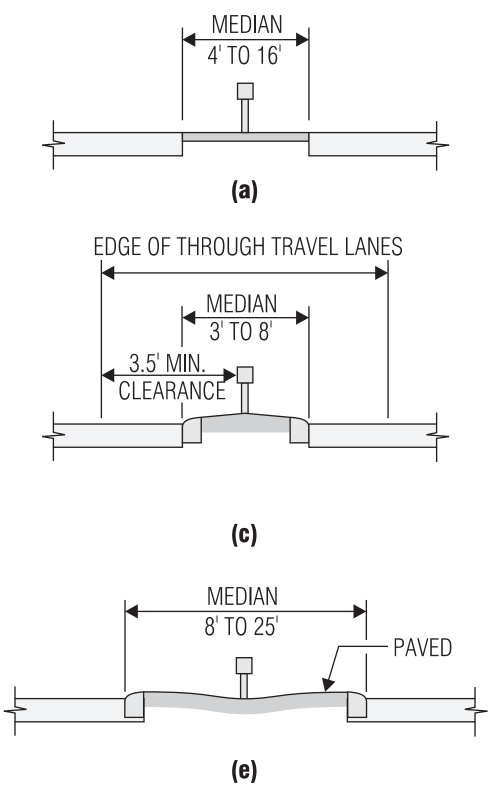

Fig. 16.10 Cross sections of a highway with medians (*a*) paved flush; (*b*) with swale and paved flush (maximum slope, 1:6) when median width exceeds 36 ft; otherwise, paved and incorporating a median barrier; (*c*) raised, curbed, and crowned, with 3-ft width when optional median barrier is installed; (*d*) natural ground between independent roadways; (*e*) raised, curbed, and depressed toward median barrier.

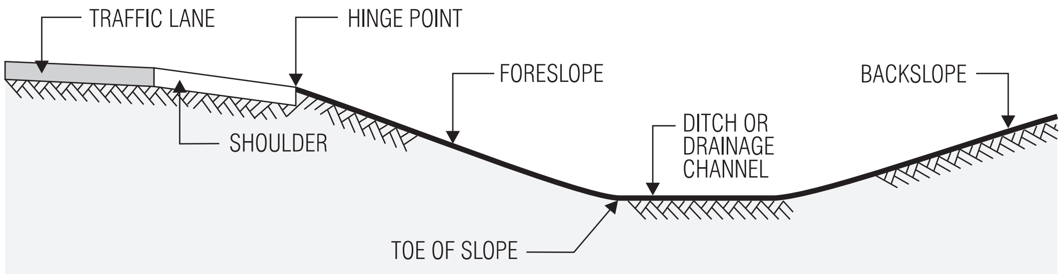

Fig. 16.11 Typical elements of a roadside.

zone width of 10 ft. For highways in an urban environment where space for clear zones is at a premium, a minimum clear-zone width of 1.5 ft should be maintained beyond the face of curbs.

16.10.2 Side Slopes

These provide stability for the roadway and give drivers of errant vehicles an opportunity to regain control. Composition to be used for side slopes depends on the geographic region and availability of materials. Rounding and blending of the slopes with the existing topography will enhance highway safety and aesthetics.

In Fig. 16.11, the hinge point is identified as the intersection of the extreme edge of the shoulder and the foreslope. From a safety standpoint, the hinge point is critical, since it is possible for drivers to lose control of their vehicles (and even become airborne) at this location. The foreslope and toe of slope also are critical because of potential safety hazards when vehicles attempt a recovery after leaving the roadway.

To help minimize these and other potential unsafe conditions, the hinge point and slopes are rounded, thus reducing the chance of an errant vehicle's becoming airborne. In addition, slopes should not be steeper than 3:1 and preferably not steeper than 4:1, especially for foreslopes, the region where vehicle recovery is likely to take place. When steeper slopes are demanded by specific site characteristics, a roadside barrier should be installed.

For backslopes, the slope should be 3:1 or flatter to facilitate operation of maintenance equipment, such as mowers. When site constraints mandate slopes steeper than 2:1, for example, in urban areas where real estate is at a premium, installation of retaining walls should be investigated.

16.10.3 Berms

These are used along rural highways on embankments or around islands to retain drainage in the shoulder and inhibit erosion of the side slope. A berm is a raised shelf that can be formed of plain earth and sodded or paved with road- or plant-mix bituminous material.

16.10.4 Fences

These are often installed along a highway to limit or control access to the highway right-of-way by pedestrians or vehicles. Fencing can also be used to prevent indiscriminate crossing of a median by vehicles, reduce headlight glare, and prevent animals from entering the highway. For these purposes, a chain-link fence 6 ft high is generally erected. In rural areas, however, a 4-ft-high farm fence is frequently used. In many instances, rural fencing is employed to prevent entry of livestock onto a highway. Fences also are installed on bridges to prevent vandals on the bridges from throwing objects down onto underpasses and causing accidents. When control of pedestrian access to a highway is the principal concern, a thick hedge may be planted to control access to the highway.

16.10.5 Noise Barriers

Incorporation of barriers to reduce the effects of noise on occupied areas adjacent to a highway, although often expensive, may be necessary. The noise generated by large volumes of traffic can severely impact residential and other properties where people live and work. Sources of highway-traffic noise include vehicle motors, vehicle exhaust, aerodynamic effects, and interaction of tires and roadway surface. For a major highway, design, beginning with the preliminary design stage, should take into account the anticipated noise levels and the type of noise barrier, if any, that will be required.

Noise barriers are sound-absorbing or sound-reflecting walls. They often are fabricated of concrete, wood, metal, or masonry. The type selected should be aesthetically pleasing and blend well with the surrounding topography. Local availability of materials or components and applicable standards often play a critical role in the selection of types of noise barriers.

Design and installation of noise barriers for a highway should conform with the general geometric design constraints of the highway. The barriers should be set as much as possible away from the highway and allow proper sight distance for drivers. When noise barriers are placed close to traffic, it may be necessary to erect protective barriers with the noise barriers.

As an alternative to employment of noise barriers, there are other ways to control the effects of noise on adjacent properties. One method is to depress the highway below the level of adjacent buildings. Another possibility is elevation of the highway on an embankment or bridge above the level of adjacent buildings. To further limit noise, shrubs and trees may be planted or ground covers placed between the highway and adjoining properties.

16.10.6 Roadside Drainage Channels

A drainage channel is often incorporated in a roadside to collect and convey surface water for drainage away from the roadbed. To perform this function, drainage channels should be sized for both design runoff and excessive storm water flows.

A drainage channel usually is a ditch formed by shaping the roadside ground surface (Fig. 16.11). From a hydraulic standpoint, the best drainage channel is the one with the steepest sides. Therefore, a balance between drainage needs and the need for flatter slopes must be achieved (Art. 16.10.2).

Drainage channels should be located to avoid creation of a hazard to errant vehicles. Maintenance crews should keep the channels free from debris, which can reduce the capacity of the channels. They should also ensure that the channels are not subjected to significant erosion, deposition of material, or other causes of channel deterioration.

16.11 Right-of-Way

This is the entire area needed for construction, drainage, and maintenance of a highway as well as for access to and exit from the highway. Achievement of many of the desirable design features discussed in Art. 16.10, such as flatter slopes and proper placement of drainage facilities, is facilitated by procurement of sufficient right-of-way. In addition, acquisition of large right-of-way allows future highway expansion to accommodate larger traffic volumes. As a minimum, however, the size of the right-of-way acquired for a highway should be at least that required for incorporation of all elements in the design cross section and the appropriate border areas.

For estimating right-of-way required for a typical ground-level freeway, for example, the cross section may be assumed to contain 12-ft lanes, 56-ft median, 50-ft outer roadsides, 30-ft frontage roads, and 15-ft borders. The American Association of State Highway and Transportation Officials (AASHTO) recommends a width of right-of-way of about 225 ft for such a freeway with no frontage roads and 300 to 350 ft with one-way frontage roads on both sides of the through pavement. For a ground-level freeway with restricted cross section, AASHTO recommends a width of 100 to 150 ft with no frontage road and 100 to 200 ft with a two-way frontage road on one side. Different sizes of right-of-way are recommended for other types of highways (AASHTO "A Policy on Geometric Design of Highways and Streets").

16.12 Superelevation

It is desirable to construct one edge of a roadway higher than the other along curves of highways to counteract centrifugal forces on passengers and vehicles, for the comfort of passengers and to prevent vehicles from overturning or sliding off the road if the centrifugal forces are not counteracted by friction between the roadway and tires. Because of the possibility of vehicle sliding when the curved road is covered with rain, snow, or ice, however, there are limitations on the amount of superelevation that can be used.

The maximum superelevation rate to use depends on local climate and whether the highway is classified as rural or urban. Table 16.2 presents typical limits for various design speeds, min-

Table 16.2 Superelevation e, in/ft, of Pavement Width and Spiral Length, L_s, ft, for Horizontal Curves of Highway*

Degree D_c of curve	Radius of curve, ft	Vehicle design velocity, mi/h																				
		30			40			50			60			65			70			75		
		e	L_s Lanes 2	L_s Lanes 4	e	L_s Lanes 2	L_s Lanes 4	e	L_s Lanes 2	L_s Lanes 4	e	L_s Lanes 2	L_s Lanes 4	e	L_s Lanes 2	L_s Lanes 4	e	L_s Lanes 2	L_s Lanes 4	e	L_s Lanes 2	L_s Lanes 4
0° 15′	22,918	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
0° 30′	11,459	NC	0	0	NC	0	0	NC	0	0	RC	175	175	RC	190	190	RC	200	200	0.022	220	220
0° 45′	7,639	NC	0	0	NC	0	0	RC	150	150	0.022	175	175	0.025	190	190	0.029	200	200	0.032	220	220
1° 00′	5,730	NC	0	0	RC	125	125	0.021	150	150	0.029	175	175	0.053	190	190	0.038	200	200	0.043	220	220
1° 30′	3,820	RC	100	100	0.021	125	125	0.030	150	150	0.040	175	175	0.046	190	200	0.053	200	240	0.080	220	290
2° 00′	2,865	RC	100	100	0.027	125	125	0.038	150	150	0.051	175	210	0.057	190	250	0.065	200	290	0.072	230	340
2° 30′	2,292	0.021	100	100	0.033	125	125	0.046	150	170	0.060	175	240	0.066	190	290	0.073	220	330	0.078	250	370
3° 00′	1,910	0.025	100	100	0.038	125	125	0.053	150	190	0.067	180	270	0.073	210	320	0.073	230	350	0.080	250	380
3° 30′	1,637	0.028	100	100	0.043	125	140	0.058	150	210	0.073	200	300	0.077	220	330	0.080	240	380	**0.080**	**250**	**380**
4° 00′	1,432	0.052	100	100	0.047	125	150	0.063	150	230	0.077	210	310	0.079	230	340	**0.080**	**240**	**360**	**D_c max = 3.0°**		
5° 00′	1,146	0.038	100	100	0.055	125	170	0.071	170	260	0.080	220	320	**0.080**	**230**	**350**	**D_c max = 3.5°**					
6° 00′	955	0.043	100	120	0.061	130	190	0.077	180	280	**0.080**	**220**	**320**	**D_c max = 4.5°**								
7° 00′	819	0.048	100	130	0.067	140	210	0.079	190	280	**D_c max = 5.0°**											
8° 00′	716	0.052	100	140	0.071	150	220	**0.080**	**190**	**290**												
9° 00′	637	0.056	100	150	0.075	160	240	**D_c max = 7.5°**														
10° 00′	573	0.059	110	160	0.077	160	240															
11° 00′	521	0.063	110	170	0.079	170	250															
12° 00′	477	0.066	120	180	0.080	170	230															
13° 00′	441	0.068	120	180	**0.080**	**170**	**250**															
14° 00′	409	0.070	130	190	**D_c max = 12.5°**																	
16° 00′	358	0.074	130	200																		
18° 00′	318	0.077	140	210																		
20° 00′	286	0.079	140	210																		
22° 00′	260	0.080	140	220																		
		0.080	**140**	**220**																		
		D_c max = 23.0°																				

*Adapted from "Highway Design Manual," New York State Department of Transportation.

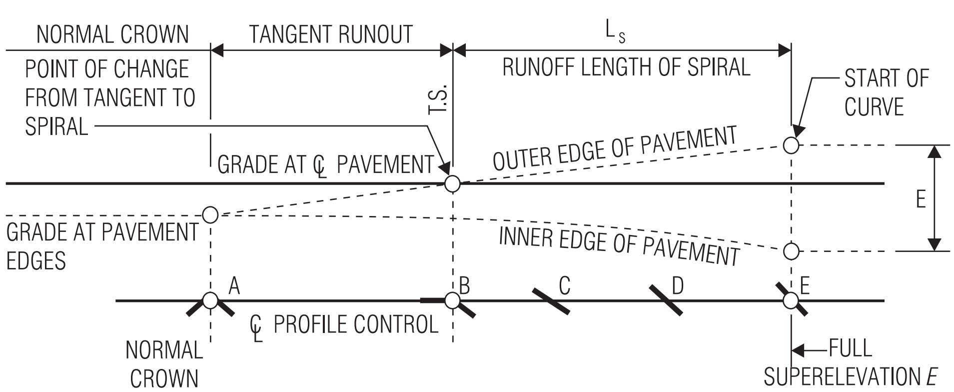

Fig. 16.12 Superelevation variations along a spiral transition curve.

imum radii, superelevation rates e, and transition spiral lengths L_s. The last is the distance over which the normal crown cross section changes to a fully banked section as the roadway alignment changes from tangent to start of a circular curve.

For the safety and comfort of drivers, provision usually is made for gradual change from a tangent to the start of a circular curve. One method for doing this is to insert a spiral curve between those sections of the roadway (Art. 16.13.3). A spiral provides a comfortable path for drivers since the radius of curvature of the spiral gradually decreases to that of the circular curve while the superelevation gradually increases from zero to full superelevation of the circular curve. A similar transition is inserted at the end of the circular curve. (An alternative is to utilize compound curves that closely approximate a spiral.) Over the length of the transition, the centerline of each roadway is maintained at profile grade while the outer edge of the roadway is raised and the inner edge is lowered to produce the required superelevation. As indicated in Fig. 16.12, typically the outer edge is raised first until the outer half of the cross section is level with the crown (point *B*). Then, the outer edge is raised further until the cross section is straight (point *C*). From there on, the entire cross section is rotated until the full superelevation is attained (point *E*). See also Art. 16.13.4.

Superelevated roadway cross sections are typically employed on curves of rural highways and urban freeways. Superelevation is rarely used on local streets in residential, commercial, or industrial areas.

Highway Alignments

Geometric design of a highway is concerned with horizontal and vertical alignment as well as the cross-sectional elements discussed in Arts. 16.2 to 16.12. Horizontal alignment of a highway defines its location and orientation in plan view. Vertical alignment of a highway deals with its shape in profile. For a roadway with contiguous travel lanes, alignment can be conveniently represented by the centerline of the roadway.

16.13 Horizontal Alignment

This comprises one or more of the following geometric elements: tangents (straight sections), circular curves (Art 16.13.2), and transition spirals (Arts. 16.12 and 16.13.3).

16.13.1 Stationing

Distance along a horizontal alignment is measured in terms of stations. A full station is defined as 100 ft and a half station as 50 ft. Station 100 + 50 is 150 ft from the start of the alignment, Station 0 + 00. A point 1492.27 ft from 0 + 00 is denoted as 14 + 92.27, indicating a location 14 stations (1400 ft) plus 92.27 ft from the starting point of the alignment. This distance is measured horizontally along the centerline of the roadway, whether it is a tangent, curve, or a combination of these.

16.13.2 Simple Curves

A simple horizontal curve consists of a part of a circle tangent to two straight sections on the horizontal alignment. The radius of a curve preferably should be large enough that drivers do not feel compelled to slow their vehicles. Such a radius, however, is not always feasible, inasmuch as the alignment should blend harmoniously with the existing topography as much as possible and balance other design considerations, such as overall project economy, sight distance, and side friction. Superelevation, usually employed on curves with sharp curvature, also should be taken into account (Art. 16.12).

A curve begins at a point designated *point of curvature* or *PC*. There, the curve is tangent to the straight section of the alignment, which is called a *tangent* (Fig. 16.13). The curve ends at the *point of tangency PT*, where the curve is tangent to another straight section of the alignment. The angle Δ formed at *PI, the point of intersection* of the two tangents, is called the *interior angle* or *intersection angle*.

The curvature of a horizontal alignment can be defined by the radius *R* of the curve or the degree of curve *D*. **One degree of curve** is the central angle that subtends a 100-ft arc (approximately a 100-ft chord). The degree of a curve is given by

$$D = \frac{5729.8}{R} \tag{16.1}$$

Values for the minimum design radius allowable for normal crown sections are presented in Table 16.3.

The length of the tangent *T* (distance from *PC* to *PI* or *PI* to *PT*) can be computed from

$$T = R \tan \frac{\Delta}{2} \tag{16.2}$$

The external distance E measured from PI to the curve on a radial line is given by

$$E = R\left(\sec\frac{\Delta}{2} - 1\right) \qquad (16.3)$$

The middle ordinate distance M extends from the midpoint B of the chord to the midpoint A of the curve.

$$M = R\left(1 - \cos\frac{\Delta}{2}\right) \qquad (16.4)$$

The length of the chord C from PC to PT is given by

$$C = 2R\sin\frac{\Delta}{2} = 2T\cos\frac{\Delta}{2} \qquad (16.5)$$

Table 16.3 Maximum Curvature for Normal Crown Section*

Design speed, mi/h	Average running speed, mi/h	Maximum degree of curve	Minimum curve radius, ft
20	20	3° 23′	1,700
30	28	1° 43′	3,340
40	36	1° 02′	5,550
50	44	0° 41′	8,320
55	48	0° 35′	9,930
60	52	0° 29′	11,690
65	55	0° 26′	13,140
70	58	0° 23′	14,690

*Adapted from "A Policy on Geometric Design of Highways and Streets," American Association of State Highway and Transportation Officials.

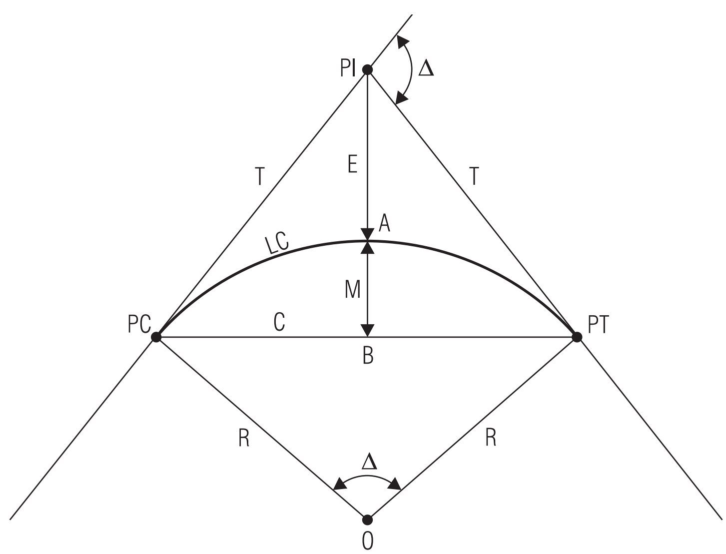

Fig. 16.13 Circular curve starting at point PC on one tangent and ending at PT on a second tangent that intersects the first one at PI. Curve radius is R and chord distance between PC and PT is C. Length of arc is LC. Tangent distance is T.

The length L of the curve can be computed from

$$L = \frac{\Delta \pi R}{180} = \frac{100\Delta}{D} \tag{16.6}$$

where Δ = intersection angle, degrees.

16.13.3 Transition (Spiral) Curves

On starting around a horizontal circular curve, a vehicle and its contents are immediately subjected to centrifugal forces. The faster the vehicle enters the circle and the sharper the curvature, the greater the influence on vehicles and drivers of the change from tangent to curve. For example, depending on the friction between tires and road, vehicles may slide sideways, especially if the road is slick. Furthermore, drivers are uncomfortable because of the difficulty of achieving a position of equilibrium. A similar condition arises when a vehicle leaves a circular curve to enter a straight section of highway. To remedy these conditions, especially where high-speed traffic must round sharp curves, a transition curve with a constantly changing radius should be inserted between the circular curve and the tangent. The radius of the transition curve should vary gradually from infinity at the tangent to that of the circular curve. Along the transition, superelevation should be applied gradually from zero to its full value at the circular curve.

An **Euler spiral** (also known as a **clothoid**) is typically used as the transition curve. The gradual change in radius results in a corresponding gradual development of centrifugal forces, thereby reducing the aforementioned adverse effects. In general, transition curves are used between tangents and sharp curves and between circular curves of substantially different radii. Transition curves also improve driving safety by making it easier for vehicles to stay in their own lanes on entering or leaving curves. When transition curves are not provided, drivers tend to create their own transition curves by moving laterally within their travel lane and sometimes the adjoining lane, a hazardous maneuver. In addition, transition curves provide a more aesthetically pleasing alignment, giving the highway a smooth appearance without noticeable breaks at the beginning and end of circular curves.

The minimum length L, ft, of a spiral may be computed from

$$L = \frac{3.15V^3}{RC} \tag{16.7}$$

where V = vehicle velocity, mi/h

R = radius, ft, of the circular curve to which the spiral is joined

C = rate of increase of radial acceleration

An empirical value indicative of the comfort and safety involved, C values often used for highways range from 1 to 3. (For railroads, C is often taken as unity 1.) Another, more practical, method for calculating the minimum length of spiral required for use with circular curves is to base it on the required length for superelevation runoff (Art. 16.13.4).

16.13.4 Superelevation Runoff L_s

This is the length of highway required to alter the cross section of a roadway from normal crown to fully superelevated, or vice versa (Fig. 16.12). Table 16.2 lists values of L_s for two- and four-lane pavements and for various design velocities. The table is based on the assumption that the centerline of each roadway is maintained at profile grade while the outer edge is raised and the inner edge lowered to create the required superelevation. The superelevation runoff is effected uniformly to provide both comfort and safety. AASHTO recommendations for superelevation runoff may differ somewhat from those given in Table 16.2. Also, high-type alignments may require longer runoffs, and while the runoff for wide pavements is greater than that for two-lane pavements, there are no generally accepted length ratios.

A certain amount of prudence should be exercised in design in the use of any of the receding criteria. For example, if a highway is located in a cut with a relatively flat profile, lowering of the inner edge may result in a sag from which surface water cannot be properly drained. To prevent this condition, superelevation should be achieved through raising of the outer edge. This will require elevating this edge twice the distance needed when the inner edge is lowered. As another example, if superelevation is employed on a divided highway, an undesirable condition may arise if superelevation is applied by rotating about its centerline the pavement on each side of the median. The two sides of the median will end up at substantially different elevations. A better alternative is to rotate each pavement about the roadway edge adjoining the median.

16.13.5 Passing Sight Distance

On two-lane highways, drivers should be provided at intervals safe opportunities to pass slow-moving vehicles. Failure to do so increases the risk of head-on collisions and tends to decrease highway traffic capacity. To permit safe passing, a driver must be able to see far enough ahead to be certain that there is no danger of collision with an oncoming vehicle or an obstruction in the highway. Table 16.4 lists minimum sight distances that can serve as a guide in designing highway alignment.

Table 16.4 Minimum Passing Sight Distances for Design of Two-Lane Highways

Design speed, mi/h	Assumed passed-vehicle speed, mi/h*	Minimum passing sight distance, ft
30	26	1100
40	34	1500
50	41	1800
60	47	2100
65	50	2300
70	54	2500
75	56	2600
85	59	2700

*Assumed speed of passing vehicle 10 mi/h faster than that of the passed vehicle.

16.14 Vertical Alignment

A vertical alignment defines the geometry of a highway in elevation, or profile. A vertical alignment can be represented by the highway centerline along a single tangent at a given grade, a vertical curve, or a combination of these.

16.14.1 Clearance for Bridges

When a highway is carried on a bridge over an obstruction, a minimum clearance should be maintained between the underside of the bridge superstructure and the feature crossed. AASHTO's Standard Specifications for Highway Bridges specifies an absolute minimum clearance of 14 ft and design clearance of 16 ft.

16.14.2 Vertical Curves

These are used as a transition where the vertical alignment changes grade, or slope. Vertical curves are designed to blend as best as possible with the existing topography, consideration being given to the specified design speed, economy, and safety. The tangents to a parabolic curve, known as grades, can affect traffic in many ways; for example, they can influence the speed of large tractor trailers and stopping sight distance.

Although a circular curve can be used for a vertical curve, common practice is to employ a parabolic curve. It is linked to a corresponding horizontal alignment by common stationing. Figure 16.14 shows a typical vertical curve and its constituent elements.

A curve like the one shown in Fig. 16.14 is known as a crest vertical curve; that is, the curve crests like a hill. If the curve is concave, it is called a sag vertical curve; that is, the curve sags like a valley. As indicated in Fig. 16.14, the transition starts on a tangent at *PVC*, point of vertical curvature, and terminates on a second tangent at PVT, point of vertical tangency. The tangents, if extended, would meet at *PVI*.

The basic properties of a parabolic vertical curve are derived from an equation of the form $y = ax^2$ The rate of grade change r, percent per station of curve length, is

$$r = \frac{g_2 - g_1}{L} \tag{16.8}$$

where g_1 = grade, percent, at PVC, shown positive (upward slope) in Fig. 16.14

g_2 = grade, percent, at PVT, shown negative (downward slope)

L = length, stations, of vertical curve

If a curve has a length of 700 ft, $L = 7$. If grade g_1 at *PVC* were 2.25% and grade g_2 at *PVT* were −1.25%, the rate of change would be $r = (-1.25 - 2.25)/7 = -0.50\%$ per station.

A key point on a vertical curve is the **turning point**, where the minimum or maximum elevation on a vertical curve occurs. The station at this point may be computed from

$$x_{TP} = \frac{-g_1}{r} \tag{16.9}$$

The middle ordinate distance e, the vertical distance from the *PVI* to the vertical curve, is given by

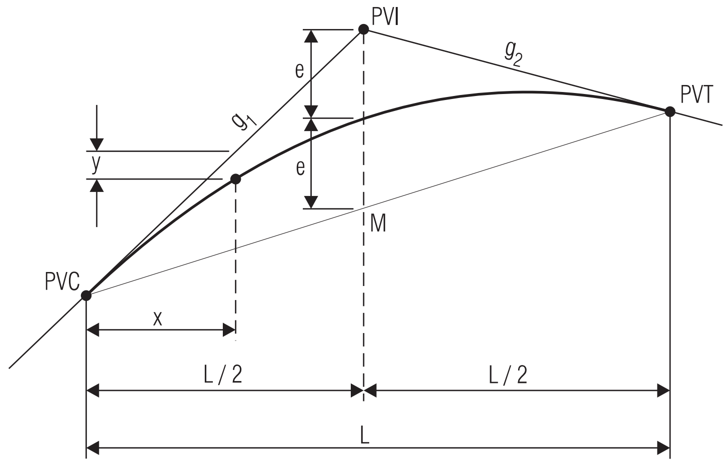

Fig. 16.14 Parabolic vertical curve starting at point *PVC* on one tangent and terminating at *PT* on a second tangent that intersects the first one at *PVI* at a distance *e* above the curve.

$$e = \frac{(g_1 - g_2)L}{8} \tag{16.10}$$

For the layout of a vertical curve in the field, it is necessary to know the elevations at points along the curve. From the equation of a parabola, the elevation H_x, ft, of the curve at a distance x, stations, from the *PVC* may be computed from

$$H_x = H_1 + g_1 x + \frac{rx^2}{2} \tag{16.11}$$

where H_1 = elevation of the *PVC*. The last term of the equation $rx^2/2$ is the vertical offset of the curve from a point on the tangent to the curve at a distance x, stations, from *PVC*.

16.14.3 Stopping Sight Distance

This is the length of roadway needed between a vehicle and an arbitrary object (at some point down the road) to permit a driver to stop a vehicle safely before reaching the obstruction. This is not to be confused with passing sight distance, which AASHTO defines as the "length of roadway ahead visible to the driver." (Art. 16.13.5). Figure 16.15 shows the parameters governing stopping sight distance on a crest vertical curve.

The minimum stopping sight distance is computed for a height of eye (driver eye height) of 3.50 ft and a height of object (obstruction in roadway) of 6 in. The stopping distance on a level roadway comprises the distance over which a vehicle moves during the brake reaction time, the time it takes a driver to apply the brakes on sighting an obstruction, and the distance over which the vehicle travels before coming to a complete stop (braking distance). Table 16.5 lists approximate stopping sight distances on a level roadway for various design speeds and wet pavements. If the vehicle is traveling uphill, the braking distance is less, because gravity aids in slowing the vehicle. For downhill movement, braking distance is more.

A general rule of thumb is that the longer a vertical curve, the larger the safe stopping sight distance may be. Long curves, however, may be too costly to construct. Therefore, a balance should be reached between economy and safety without jeopardizing safety.

For crest vertical curves AASHTO defines the minimum length L_{min}, ft, of crest vertical curves based on a required sight distance S, ft, as that given by Eqs. (16.12) to (16.15).

$$L_{min} = \frac{AS^2}{100\left(\sqrt{2H_1} + \sqrt{2H_2}\right)^2} \quad S < L \tag{16.12}$$

When eye height is 3.5 ft and object height is 0.5 ft,

$$L_{min} = \frac{AS^2}{1329} \quad S < L \tag{16.13}$$

Also, for crest vertical curves,

$$L_{min} = 25 - \frac{200\left(\sqrt{H_1} + \sqrt{H_2}\right)^2}{AS^2} \quad S > L \tag{16.14}$$

When eye height is 3.5 ft and object height 0.5 ft,

Table 16.5 Design Controls for Vertical Curves Based on Stopping Sight Distance*

Design speed, mi/h	Average speed for condition, mi/h	Coefficient of friction f	Stopping sight distance (rounded for design), ft	Rate of vertical curvature K, ft per percent of A: For crest curves, Computed	For crest curves, Rounded for design	For sag curves, Computed	For sag curves, Rounded for design
20	20 – 20	0.40	125 – 125	8.6 – 8.6	10 – 10	14.7 – 14.7	20 – 20
25	24 – 25	0.38	150 – 150	14.4 – 16.1	20 – 20	21.7 – 23.5	30 – 30
30	28 – 30	0.35	200 – 200	23.7 – 28.8	30 – 30	30.8 – 35.3	40 – 40
35	32 – 35	0.34	225 – 250	35.7 – 46.4	40 – 50	40.8 – 48.6	50 – 50
40	36 – 40	0.32	275 – 325	53.6 – 73.9	60 – 80	53.4 – 65.6	60 – 70
45	40 – 45	0.31	325 – 400	76.4 – 110.2	80 – 120	67.0 – 84.2	70 – 90
50	44 – 50	0.30	400 – 475	106.6 – 160.0	110 – 160	82.5 – 105.6	90 – 110
55	48 – 55	0.30	450 – 550	140.4 – 217.6	150 – 220	97.6 – 126.7	100 – 130
60	52 – 60	0.29	525 – 650	189.2 – 302.2	190 – 310	116.7 – 153.4	120 – 160
65	55 – 65	0.29	550 – 725	227.1 – 394.3	230 – 400	129.9 – 178.6	130 – 180
70	58 – 70	0.28	625 – 850	282.8 – 530.9	290 – 540	147.7 – 211.3	150 – 220

*Adapted from "A Policy on Geometric Design of Highways and Streets," American Association of State Highway and Transportation Officials.

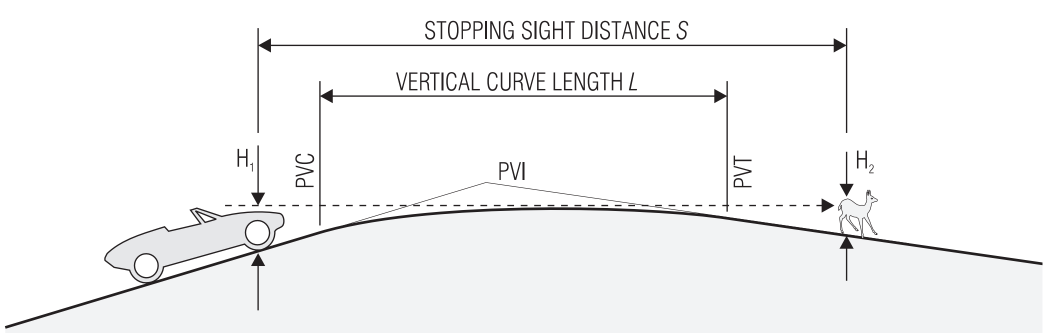

Fig. 16.15 Stopping sight distance on a crest vertical curve.

$$L_{\min} = 25 - \frac{1329}{AS^2} \quad S > L \tag{16.15}$$

where A = algebraic difference in grades, percent, of the tangents to the vertical curve

H_1 = eye height, ft, above the pavement

H_2 = object height, ft, above the pavement

Design controls for vertical curves can be established in terms of the rate of vertical curvature K defined by

$$K = \frac{L}{A} \tag{16.16}$$

where L = length, ft, of vertical curve and A is defined above. K is useful in determining the minimum sight distance, the length of a vertical curve from the *PVC* to the turning point (maximum point on a crest and minimum on a sag). This distance is found by multiplying K by the approach gradient.

Table 16.5 lists recommended values of K for various design velocities and stopping sight distances for crest and sag vertical curves.

Highway Drainage

Proper drainage is a very important consideration in design of a highway. Inadequate drainage facilities can lead to premature deterioration of the highway and the development of adverse safety conditions such as hydroplaning. It is common, therefore, for a sizable portion of highway construction budgets to be devoted to drainage facilities.

In essence, the general function of a highway drainage system is to remove rainwater from the road and water from the highway right-of-way. The drainage system should provide for the drainage conditions described in Arts. 16.16 and 16.17.

16.15 Storm Frequency and Runoff

Storm frequency refers to the chance that a given intensity of rainfall will occur within a specific span of years. It is determined from historical data that indicate that a particular intensity of rainfall can be expected once in N years. A drainage system designed for such an intensity is intended to be capable of withstanding an N-year storm, runoff, or flood. A 25-year storm, for example, represents a 1 in 25 probability that the drainage system will have to accommodate such an intensity. This does not mean that every 25 years a certain storm of this magnitude will occur. It is possible that such a storm will not occur at all during any 25-year period. It is also possible, however, that two or more such storms will take place in a single year. The odds of this happening, though, are relatively small.

For highways, cross drains (small culverts) passed under major highways to carry the flow from defined watercourses are typically designed to accommodate a 25-year storm. Larger culverts and bridges on major highways are designed with capacity for 100-year storms. For nonmajor highways, the storm used for design can range from a 10- to 50-year storm, depending on the highway size and traffic volume expected.

Runoff Determination ■ The amount of runoff to be used for design of surface drainage can be determined through physical stream-flow measurements or through the use of empirical formulas. A common approach is to utilize the rational method described in Art. 21.39 (also known as the Lloyd-Davies method in the United Kingdom). While this approach gives reasonable answers in most urban areas, care must be taken when applying the rational method in rural areas. Runoff for rural and large watershed areas is much more difficult to estimate accurately than the runoff in urban environments. Typically, for determination of runoff, a large watershed is divided into several smaller watershed areas, from which runoff flows to various inlets or waterways. In general, conservative design values of runoff can be determined for drainage areas of 100 acres or less. Some designers, however, have used 200-acre and even 500-acre maximum values.

16.16 Surface Drainage

Provision must be made for removal of water, from rain or melting snow, or both, that falls directly on a road or comes from the adjacent terrain. The road should be adequately sloped to drain the water away from the travel lanes and shoulders and then directed to drainage channels in the system, such as natural earth swales, concrete gutters, and ditches, for discharge to an adjacent body of water. The channels should be located and shaped to minimize the potential for traffic hazards and accommodate the anticipated storm-water flows. Drainage inlets should be provided as needed to prevent ponding and limit the spread of water into traffic lanes.

16.16.1 Surface Drainage Methods

For rural highways on embankments, runoff from the roadway should be allowed to flow evenly over the side slopes and then spread over the adjacent terrain. This method, however, can sometimes adversely impact surrounding land, such as farms. In such instances the drainage should be collected, for example, in longitudinal ditches and then conveyed to a nearby watercourse.

When a highway is located in a cut, runoff may be collected in shallow side ditches. These typically have a trapezoidal, triangular, or rounded cross section and should be deep enough to drain the pavement subbase and convey the design-storm flow to a discharge point. Care should be taken to design the ditches so that the toe of adjoining sloping fill does not suffer excessive erosion. For larger water flows than the capacity of a shallow ditch, paved gutters or drainpipes with larger capacities will have to be used.

In urban environments and built-up areas, use of roadside drainage channels may be severely limited by surrounding land uses. In most instances, the cost of acquiring the necessary right-of-way to implement such drainage facilities is prohibitive. For highways on embankments, a curb or an earth berm may be constructed along the outer edge of the roadway to intercept runoff and divert it to inlets placed at regular intervals. The inlets, in turn, should be connected to storm sewers that convey the water to points of disposal. In an urban area, it may be necessary to construct storm sewers of considerable length to reach the nearest body of water for discharge of the runoff.

16.16.2 Inlets

These are parts of a drainage system that receive runoff at grade and permit the water to flow downward into underground storm drains. Inlets should be capable of passing design floods without clogging with debris. The entrance to inlets should be protected with a grating set flush with the surface of gutters or medians, so as not to be a hazard to vehicles. There are several types of inlets.

A **drop inlet** is a box-type structure that is located in pipe segments of a storm-water collection system and into which storm water enters from the top. Most municipal agencies maintain design and construction standards for a wide variety of inlets, manholes, and other similar structures, but some large structures may require site-specific design.

A **curb inlet** consists of a vertical opening in a curb through which gutter flow passes. A **gutter inlet** is a horizontal opening in the gutter that is protected by a single grate or multiple grates through which the gutter flow passes. A **combination inlet** consists of both gutter and curb inlets with the gutter inlet placed in front of the curb inlet.

Inlet spacing depends on the quantity of water to be intercepted, shape of ditch or gutter conveying the water, and hydraulic capacity of the inlet.

16.16.3 Storm Sewers

These are underground pipes that receive the runoff from a roadside inlet for conveyance and discharge into a body of water away from the road. Storm sewers are often sized for anticipated runoff and for pipe capacity determined from the Manning formula (Art. 21.9).

In general, changes in sewer direction are made at inlets, catch basins, or manholes.

The manholes should provide maintenance access to sewers at about every 500 ft.

A storm sewer system for a new highway should be connected to an existing drainage system, such as a stream or existing storm sewer system. If a storm sewer is to connect to a stream, the downstream conditions should be investigated to ensure that the waterway is adequate and that the new system will not have an adverse environmental impact. If the environmental impact is not acceptable, it will be necessary to study possible improvements to downstream outlets to accommodate the additional flow or to make the drainage scheme acceptable to local officials in some other fashion.

16.16.4 Open Channels

As indicated in Art. 16.16.1, side ditches may be used to collect runoff from a highway located in a cut. The ditches may be trapezoidal or V-shaped. The trapezoidal ditch has greater capacity for a given depth. Most roadway cross sections, however, include some form of V-shaped channel as part of their cross-sectional geometry. In most instances, it is not economical to vary the size of these channels. As a result, this type of channel generally has capacity to spare, since a normal depth must be maintained to drain the pavement subbase courses.

When steep grades are present, the possibility of ditch erosion becomes a serious consideration. Erosion can be limited by lining the channel with sod, stone, bituminous or concrete paving, or by providing small check dams at intervals that depend on velocity, type of soil, and depth of flows.

Linings for roadside channels are typically classified as either rigid or flexible. Paved and concrete linings are examples of rigid linings. Rock (riprap) and grass linings are examples of flexible linings. While rigid linings are better at limiting erosion, they often permit higher water velocities since they are smoother than flexible linings.

Roadside channels are often sized for anticipated runoff and for open-channel flow computed from the Manning equation (Art. 21.24). This equation includes a roughness coefficient n that may range from as low as 0.02 for concrete to 0.10 for thick grass. Flow in open channels is discussed in Arts. 21.23 to 21.33, which deal with hydraulic jump and normal, subcritical, and supercritical flow. Flow down gentle slopes is likely to be subcritical whereas flow down steep slopes may be supercritical. When the water depth is greater than the critical depth, subcritical flow occurs. Conversely, when the depth of water is less than the critical depth, supercritical flow occurs. The abrupt transition from subcritical to supercritical flow takes the form of a hydraulic jump.

Open channels should be designed to avoid supercritical flow. The reason for this is that water moving through a channel at high speeds can generate waves that travel downstream and cause water to overtop the sides of the channel and scour the downstream outlet. To limit the effects of scour at the outlet, energy dissipators may be incorporated in the channel. An energy dissipator may be a drop structure that alters the slope of the channel from steep to gentle. Alternatively, roughness elements, such as blocks and sills, can be placed in the channel to increase resistance to flow and decrease the probability of hydraulic jump's occurring.

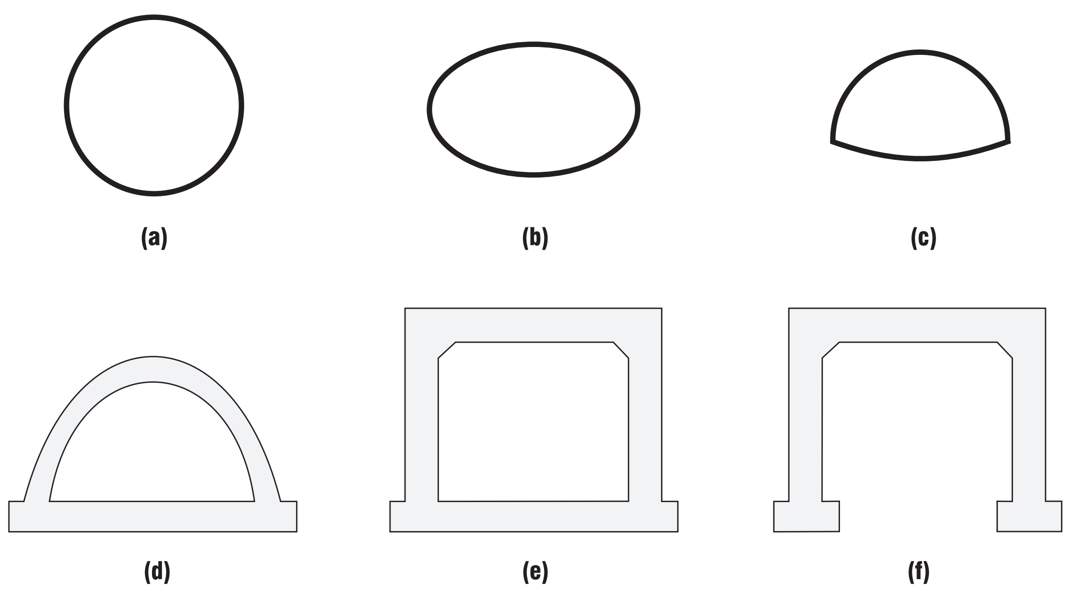

Fig. 16.16 Culvert cross sections: (*a*) circular pipe, usually concrete, corrugated metal, vitrified clay, or cast iron; (*b*) elliptical pipe, generally reinforced concrete or corrugated metal; (*c*) precast concrete pipe arch; (*d*) corrugated metal or reinforced concrete arch; (*e*) reinforced concrete box culvert; (*f*) reinforced concrete bridge culvert.

16.16.5 Culverts

A culvert is a closed conduit for passage of runoff from one open channel to another. One example is a corrugated metal pipe under a roadway. Figure 16.16 shows various types of culvert cross sections and indicates material types used in highway design.

For small culverts, stock sizes of corrugated metal pipe may be used. For larger flows, however, a concrete box or multiple pipes may be needed. If the culvert foundation is not susceptible to erosion, a bridge may be constructed over the waterway (bridge culvert).

The section of a culvert passing under a highway should be capable of withstanding the loads induced by traffic passing over the culvert. Since corrugated metal pipes are flexible, they are assisted by surrounding soil in carrying gravity loads. Reinforced concrete culverts, however, have to support gravity loads without such assistance.

Empirical methods often are used for selecting and specifying culverts. With the use of data from previous experience, designers generally select small-sized culverts from standards based on the characteristics of the project to be constructed. Larger concrete arch and box-type structures, however, are designed for the specific service loads.

Culverts are generally installed in an existing channel bed since this will result in the least amount of work in modifying existing drainage conditions. To avoid extremely long culvert lengths, however, it may be necessary to relocate an existing channel.

16.17 Subsurface Drainage

Water in underlying soil strata of a highway can move upward through capillary action and water can permeate downward to the underlying soil strata through cracks and joints in the pavement. In either case, the water can cause deterioration of the roadbed and pavement. To prevent this, subsurface drainage is used to remove water from the highway subgrade and intercept underground water before it flows to the subgrade. Although design of subsurface drainage systems depends on the specific geometry, topography, and subsurface conditions of the site to be drained, subsurface drainage facilities should be considered an integral component of the entire highway drainage system rather than treated as a separate component.

Failure to implement subsurface facilities that meet drainage requirements can lead to failure of major segments of the highway and to slope instability. Figures 16.17 to Fig. 16.19 illustrate some commonly used subgrade drainage methods.

Figure 16.17 shows an intercepting drain installed to cut off an underground flow of water

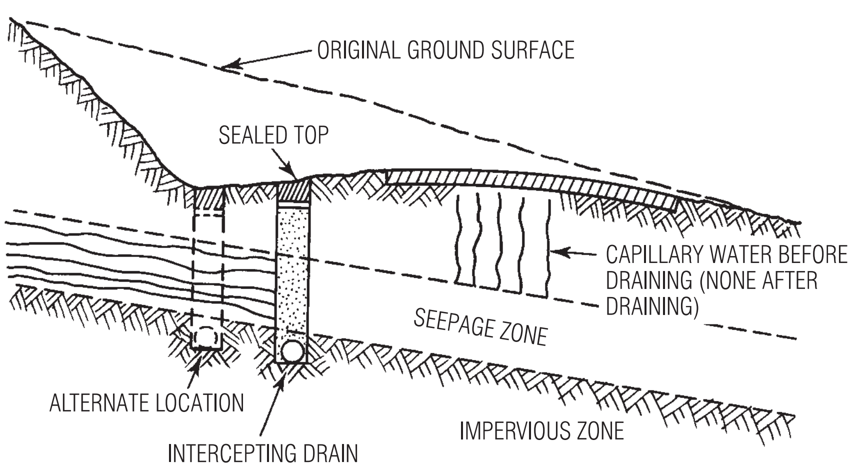

Fig. 16.17 Drain intercepts source of supply of harmful capillary and free water under a road. Top of trench is sealed to prevent silting. (*"Handbook of Drainage and Construction Products," Metal Products Division, Armco Steel Corp.*)

to prevent it from seeping into the subgrade of a road. The top of the trench is sealed to prevent silting. In Fig. 16.18, drains are shown employed on both sides of a road to remove surface water that may be trapped when a pervious base is laid over a relatively impervious subgrade. When this detail is used, the longitudinal base drains should be outletted at convenient points, which may be 100 ft apart or more. On steep slopes, lateral drains may be added under the pavement.

Figure 16.19 shows a typical bedding and backfill detail for a pipe underdrain. It is constructed by digging a trench to a specified depth, placing a pipe in the trench, and then backfilling the trench with a porous, granular material. The pipes are generally fabricated of perforated corrugated

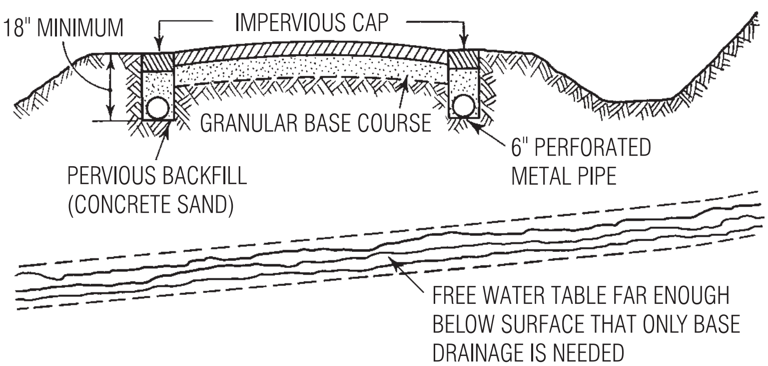

Fig. 16.18 Drains remove surface water that may be trapped when a pervious base is laid over a relatively impervious subgrade. On steep slopes, lateral may be added under the pavement. Longitudinal base drains should be outletted at convenient points, which may be 100 ft or more apart. (*"Handbook of Drainage and Construction Products," Metal Products Division, Armco Steel Corp.*)

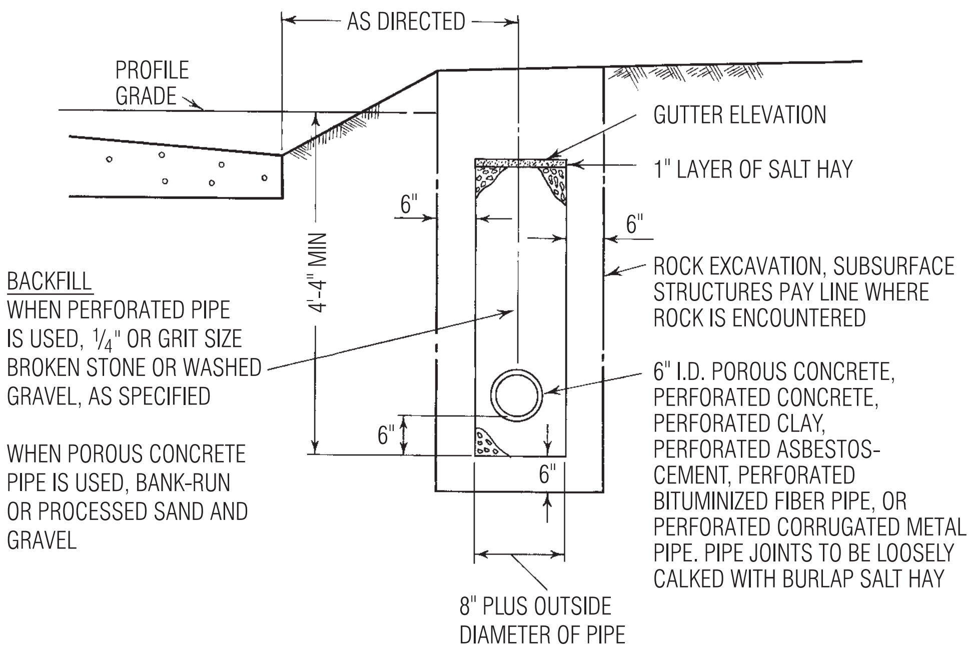

Fig. 16.19 Underdrain detail.

metal pipe, vitrified clay, or porous concrete. Sizing of pipes is typically based on previous experience, but large projects may require site-specific design.

Road Surfaces

Roads may be paved with a durable material, such as portland cement concrete or bituminous concrete, or untreated. Pavement classifications and skid resistance are discussed in Art. 16.4. The economic feasibility of many types of road surfaces depends heavily on the costs and availability locally of suitable materials.

16.18 Untreated Road Surfaces

An untreated road surface is one that utilizes untreated soil mixtures composed of gravel, crushed rock, or other locally available material, such as volcanic cinders, blast furnace slag, lime rock, chert, shells, or caliche. Such roads are sometimes used where traffic volume is low, usually no more than about 200 vehicles per day. Should larger traffic volumes develop in the future, the untreated road surface can be used as a subgrade for a higher class of pavement.

To withstand abrasion from superimposed traffic loads, a well-graded coarse aggregate (retained on No. 10 sieve) combined with sand should be used. This mixture provides a tight, water-resistant surface with interlocking aggregate that resists shearing forces. To limit deformation, sufficient binding material, such as clay, may be added to bind the aggregates. Excessive use of clay, however, can lead to surface dislocation brought on by expansion when high moisture is present.

Gravel roads are often used during staged highway construction. Staged construction allows for construction of a project in two or more phases. A dry gravel surface can serve as a temporary road for one phase while construction proceeds on another.

The initial cost of untreated surfaces is often very low compared with that of other types of surfaces. Long-term cost of the roadway may be high, however, because frequent maintenance of the surface may be required. The principal concern in maintenance of untreated road surfaces is providing a smooth surface. Smoothness may be accomplished by blading the surface of the road with a motor grader, drag, or similar device. The roadway cross slopes also need to be maintained; otherwise ponding and other associated drainage problems can occur.

16.19 Stabilized Road Surfaces

Controlled mixtures of native soil and an additive, such as asphalt, portland cement, calcium chloride, or sand-clay, can be used to form a stabilized road. Such roads can also serve as a base course for certain types of pavements.

16.19.1 Sand-Clay Roads

Sand-clay roads are composed of a mixture of clay, silt, fine and coarse sand, and, ideally, some fine gravel. This type of road is frequently used in areas where coarse gravel is not readily available. The thickness of this type of roadway is typically 8 in or more. Construction and maintenance of sand-clay roads are similar to that described in Art. 16.18 for untreated road surfaces. The economic feasibility of sand-clay roads is greatly dependent on the availability of suitable materials.

16.19.2 Stabilization with Calcium Chloride

Calcium chloride ($CaCl_2$) is a white salt with the ability to absorb moisture from the air and then dissolve in the moisture. These properties make it an excellent stabilizing agent and dust palliative. For the latter purpose, calcium chloride is most effective when the surface soil binder is more clayey than sandy.

When calcium chloride is used as a stabilizing agent on an existing surface course, the existing roadway surface should be scarified and mixed with about ½ lb/yd^2 of calcium chloride per inch of depth. For this process to be successful, however, adequate moisture must be present.

The surface of calcium chloride–treated roads is maintained by blading with a motor grader, drag, or similar device. While, under normal conditions, calcium chloride–treated roads generally require less maintenance than untreated surfaces, they require blading immediately after rain. In dry periods, a thin layer of calcium chloride should be applied in order to maintain moisture. During extended dry periods, the road surface may require patching.

Calcium chloride often is used as a deicing agent on pavements and can cause corrosion of the metal bodies of vehicles. Similarly, when used in stabilized roads, calcium chloride can corrode the metal of vehicles, but it also can have adverse environmental effects, such as contamination of groundwater. Accordingly, calcium chloride should be used advisably as a stabilizing and dust control agent.

16.19.3 Stabilization with Portland Cement

Untreated road surfaces can be stabilized by mixing the existing road surface with portland cement if the clay content in the soil is favorable for this type of treatment. A general constraint to stabilization with portland cement is that the soils in the road surface contain less than 35% clay. The required rate of application of cement varies with soil classification and generally ranges from 6 to 12% by volume. The roadway surface to be treated should be scarified to accommodate a treated depth of about 6 in. The cement should be applied uniformly to the loose material, brought to the optimum moisture content, and then lightly rolled. The quality of soil-cement surfaces can be enhanced by mixing the soils, cement, and water in a central or traveling mixing plant, then rolling the mixture after it has been placed on the road.

16.19.4 Stabilization with Asphalt

Various asphalt surface treatments can be utilized to stabilize untreated road surfaces. The process consists of application of asphalt, then aggregate uniformly distributed, and rolling. For double, triple, or other multiple surface treatments, the process is repeated several times. This type of stabilization often is used for roads with low design speeds. Surface treatment with bituminous material should not be expected to accommodate high-speed traffic since vehicles traveling at high speeds tend to dislodge the loose aggregate.

For good results in stabilization with asphalt, at the time of application the temperature should be above 40°F, there should be no rain, and the existing road surface should be dry and well compacted. Also, the quantity and viscosity of the asphalt should be in proper relationship with the temperature, size, and quantity of the aggregate used.

For use as a dust palliative, liquid asphalt may be applied at a rate of 0.1 to 0.5 gal/yd^2. This process is typically referred to as road oiling. This type of dust palliative treatment is often used as a preliminary to progressive improvement of low-type roadways.

16.20 Macadam Road Surfaces and Base Courses

Macadam pavements are derivatives of one of the oldest types of road surfaces. They were originally developed by Scottish road builder John Louden MacAdam (1756–1836). Used as both a road surface and base course, macadam pavements are usually classified as waterbound macadam or bituminous (penetration) macadam.

16.20.1 Waterbound Macadam

A waterbound macadam road is constructed with crushed stone, which is mechanically locked or keyed with stone screenings rolled into the voids and then set in place with water. For pavement thicknesses up to 9 in, a waterbound macadam pavement is typically constructed in two courses. Thicker pavements are generally constructed with three courses.

In two-course construction, the lower course is about 4 in thick and the upper course about 2 in thick. The stones in the lower course should pass a 3-in ring and be retained on a 2-in ring. The top-course stone should pass through a 2-in ring and be retained on a l-in ring. In addition to the size requirements, the stones should also be of suitable hardness.

After the base course of stones has been put down, it is rolled with a roller weighing about 10 tons or compacted with vibratory compactors. The compaction generally shrinks the course depth by roughly one-third. Therefore, if a 4-in course were desired, stone would be spread to a depth of about 6 in prior to rolling.

After the lower course has been placed and rolled, the finer top course of stone is spread on top and compacted. Next, a layer of stone chips or stone dust is shoveled over the top course and broomed into the voids as a binder. The layer is then sprinkled with water to set it. Alternate applications of binder, water, and rolling follow until a wave of mortar appears ahead of the roller. With an experienced work crew, it is possible to obtain an excellent pavement that sheds water and is suitable for light traffic in rural areas.

Waterbound macadam has been generally superseded by asphalt concrete or portland cement–treated bases. This change occurred because of advances made in plant equipment and the time-consuming nature of waterbound macadam construction. In areas where labor is readily available and inexpensive, this type of pavement may prove feasible.

16.20.2 Bituminous (Penetration) Macadam

When a bituminous material is used as the binder material in macadam, bituminous macadam is formed. After the aggregate layer is compacted, the bituminous material is applied and penetrates into the voids, binding the stone particles together. This process has led to bituminous macadam also being referred to as penetration macadam.

When the bitumen is asphalt, it is heated to about 300 to 350 °F and applied as a liquid to the compacted aggregates. The air temperature should be 40 °F or higher at the time of application and for the preceding 24 h.

A penetration-macadam top course is usually 2 to 3 in thick. It is placed on a base course about 4 in thick, similar to the lower course of waterbound macadam in which the voids are filled with small stone (Art. 16.20.1). After the base course has been rolled, excess filler is removed by stiff brooming. Next, the large stones for the top course are spread on top and the bitumen is applied. Then, while the bitumen is still warm, the large stones are keyed or choked with small stone. Excess screenings are broomed off and the surface is rolled to ensure good keying. A second application of bitumen is made and followed by a covering of stone chips or pea gravel and rolling.

16.20.3 Inverted Penetration Macadam

For inverted penetration, the process described in Art. 16.20.2 for bituminous macadam is reversed. The asphalt binder is sprayed over a prepared surface first and then covered with aggregate. This approach can be utilized for dust control, prime coat or tack coat on which a new wearing surface will be constructed, surface treatment and armor coat for temporary protection of untreated surfaces, or seal coat for leveling, strengthening, or otherwise improving existing pavements.

16.21 Surface Treatments

Various types of surface treatment are available for improving the quality of an existing pavement. Typically, a surface treatment is a thin layer of material (about ½ to ¾ in thick) applied to the surface of a road in single or multiple lifts. Surface treatments generally consist of a bituminous material applied to crushed stone by the inverted penetration method (Art. 16.20.3). Since the surface treatment is relatively thin, it is usually not intended to support loads by itself.

Surface treatments can be used to achieve a seal coat, armor coat, dust palliative, or prime or tack coat for a new wearing course. A surface treatment is applied to a granular-type base by a pressure distributor truck. This type of vehicle is equipped with a tank containing the surfacing material and a spray bar with nozzles that spread the binder over a given width of roadway.

16.21.1 Armor Coats

Named generically a surface treatment, an armor coat is applied in two or more lifts. It is generally used to provide protection to an untreated mineral surface. Armor coats are composed of a base consisting of gravel, waterbound macadam, earth, or similar material and a top course of bituminous binder covered by mineral aggregates.

16.21.2 Seal Coats

A seal coat is a coat of binder less than ½ in thick that is applied to a pavement surface and covered with fine aggregates. Seal coats are used to waterproof (seal), protect, and enhance the skid resistance of an existing pavement. They may, however, be applied in multiple lifts in a fashion similar to that described for armor coats (Art. 16.21.1).

A seal coat comprised of fine sand, emulsified asphalt, and water is known as a **slurry seal**. This type of seal coat is used to fill cracks and otherwise rejuvenate the surface of deteriorated pavements.

16.21.3 Dust Palliatives

As a pavement deteriorates, dust and fine particles can be raised by traffic. At best, this can cause a severe hindrance to visibility and at worst, extremely hazardous conditions for vehicles traveling over the road. Dust palliative surface treatments, consisting of a small quantity of a light, slow-curing oil, such as SC-70 or SC-250, may be applied to the pavement surface to control dust. The oil penetrates the pavement surface, producing a film that surrounds individual particles and binds them together.

16.21.4 Prime Coats

Before a bituminous pavement is constructed over a base of earth, gravel, or waterbound macadam, the surface is sprayed with a bitumen. The purposes are to plug capillary voids to stop upward seepage of water from the subgrade, to coat and bind dust and loose mineral particles, and to enhance adhesion between the base and surface courses. The bitumen *primes* the surface by penetrating it until the bitumen is completely absorbed.

A liquid asphalt, such as MC-30 or MC-70, or a low-viscosity road tar, such as RT-1 to RT-3, usually is used as the bitumen. Its most important characteristic is penetrating capability. Before the prime coat is applied, the existing surface should be clean and dry.

16.21.5 Tack Coats

A tack coat is used to bind together two pavement surfaces, typically a new wearing surface to an existing base surface consisting of bitumen, portland cement concrete, or other road material. Before application of the tack coat, the existing surface should be properly prepared in order for a successful bond to be formed. It is important that the existing surface be dry and free from dirt and debris. The bitumen is applied by a pressure distributor. This type of vehicle is equipped with a tank containing the surfacing material and a spray bar with nozzles that spread the bitumen. Kept free of traffic, the tack coat should be allowed to dry until it reaches an appropriate degree of stickiness to allow for proper bonding between the two layers. Then the resurfacing layer may be applied.

16.22 Flexible Pavements

Bituminous pavements are classified as flexible, whereas portland cement–concrete pavements are considered rigid. Whereas under loads, a rigid pavement acts as a beam that can span across irregularities in an underlying layer, a flexible pavement stays in complete contact with the underlying layer. A rigid pavement is designed so that it can deflect like a beam and then return to the state that existed prior to loading. Flexible pavements, however, may deform and not entirely recover when subjected to repeated loading. The decision as to which type of pavement to use depends on local availability of materials, costs, and future maintenance considerations.

16.22.1 Flexible-Pavement Courses

Figure 16.20 shows the constituent elements of a typical flexible pavement. The main components, from the bottom up, are the subgrade, subbase, granular base, and asphalt-concrete wearing surface. For Course thicknesses, see Art. 16.22.10.

Subgrade ■ This is the underlying soil that serves as the foundation for a flexible pavement. It may be native soil or a layer of selected borrow materials that are compacted to a depth below the surface of the subbase.

Subbase ■ As shown in Fig. 16.20, the subbase is the course between the subgrade and the base course. The subbase typically consists of a compacted layer of granular material, treated or untreated, or a layer of soil treated with a suitable admixture. It differs from the base course in that it has less stringent specifications for strength, aggregate types, and gradation. If the subgrade meets the requirements of a subbase course, the subbase course may be omitted. In addition to its major structural function as part of the pavement cross section, however, the subbase course can also serve many secondary

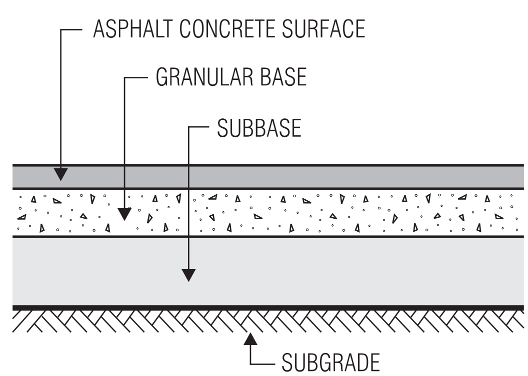

Fig. 16.20 Components of a flexible pavement.

functions, such as limiting damage due to frost, preventing accumulation of free water within or below the pavement structure, and preventing intrusion of fine-grain subgrade soils into the base courses. In rock cuts, the subbase course can also act as a working platform for construction equipment or for subsequent pavement courses. Performance of these secondary functions depends on the type of material selected for the subbase course.

Base Course ■ This is the layer of material directly under the surface course. The base course rests on the subbase or, if no subbase is provided, on the subgrade. A structural portion of the pavement, the base course consists of aggregates such as crushed stone, crushed slag, gravel and sand, or a combination of these.

Specifications for base-course materials are much more stringent than those for subbase-course materials. This is especially the case for such properties as strength, stability, hardness, aggregate types, and gradation. Addition of a stabilizing admixture, such as portland cement, asphalt, or lime, can improve the characteristics of a wide variety of materials that, if untreated, would be unsuitable for use as a base course. From an economic standpoint, such treatment is especially beneficial when there is a limited supply of suitable untreated material.

Surface Course ■ This is the uppermost layer of material in a flexible pavement. It is designed to support anticipated traffic, resist its abrasive forces, limit the amount of surface water that penetrates into the pavement, provide a skid-resistant surface, and offer a smooth riding surface. To serve these purposes, the surface course should be durable, regardless of weather conditions.

Surface courses typically consist of bituminous material and mineral aggregates that are well graded and have a maximum size of about ¾ to 1 in. Various other gradations ranging from sand (used in sheet asphalt) to coarse, open-graded mixtures of coarse and fine aggregates have been used with satisfactory results under specific conditions

16.22.2 Flexible-Pavement Design Assumptions

Flexible pavements are designed as a multilayered elastic system. Each course of a pavement is a layer with specific material properties that differ from those of the other layers and that affect the overall performance of the pavement. All layers are assumed to be infinite in the horizontal plane. The subgrade, the bottom layer, is assumed to be infinite in the vertical plane as well.

As the wheel of a vehicle passes over the pavement, compressive stresses are imposed in the surface course directly under the wheel. The surface course distributes the stresses over the base course, which, in turn, transmits them to the lower courses. The stresses are greatest at the top of the surface course and decrease toward the subgrade. Horizontal stresses exist below the wheel load also. They vary from compression (above the neutral axis of the pavement cross section) to tension (below the neutral axis). In addition, the pavement is subjected to thermal stresses.

Flexible pavements usually are designed by a method promulgated by the American Association of State Highway and Transportation Officials (AASHTO), or the Asphalt Institute, or the California Department of Transportation (Caltrans). Article 16.22.3 presents an overview of the AASHTO method.

16.22.3 AASHTO Design Method for Flexible Pavements

The AASHTO "Guide for Design of Pavement Structures" takes into account pavement performance, traffic volume, subgrade soils, construction materials, environment, drainage, reliability, life-cycle costs, and shoulder design. In essence, the design procedure is to convert the varying axle loads to a single design load and to express the traffic volume as the number of repetitions of the design axle load (Arts. 16.22.4 to 16.22.10).

16.22.4 Flexible-Pavement Performance

Pavement performance includes both the structural and functional performance of the pavement structure. Structural performance describes the ability of the pavement to support traffic loading without excessive permanent deformations, cracking, faulting, raveling, etc. Functional performance addresses the ability of the pavement to fulfill its intended functions such as maintaining a smooth and uniform riding surface.

Pavement performance is also used to describe the ability of the pavement to provide for the safety of vehicles and their passengers. An important pavement feature that impacts safety is the friction between vehicle tires and the pavement.

The influence of pavement performance in the AASHTO design method is represented by the **present serviceability index (PSI)**, which takes into account pavement roughness and distress as indicated by the extent of cracking, patching, and rut depth present. The PSI is based on a scale from 0 to 5; the higher the number the better the condition; that is, the smoother the pavement. A pavement with a PSI of 4.5, for example, is smoother (less rough) than a pavement with a PSI of 4.0. The assumption is that a smooth pavement will have a longer life than a rough one.

Two serviceability indexes are used in design of a pavement structure. One is the initial serviceability index p_i, which represents the condition of the pavement when new. The second is the terminal serviceability index p_t, which represents the minimum acceptable roughness at which stage rehabilitation is needed. AASHTO suggests the following maximum values of p_t: 2.5 or 3.0 for major highways, 2.0 for lower classifications, and 1.5 for extreme situations for low-volume roads where costs must be kept low and then on a case-by-case basis.

While deterioration and the related loss of serviceability of a pavement are related to the age of the pavement, volume of traffic, and various environmental conditions, there is no direct relationship that incorporates the combined impact of these variables. Therefore, some degree of idealization is required; for example, age may be taken as a net negative factor that reduces serviceability.

16.22.5 Traffic Loads

The effects of traffic loads are determined by the use of an equivalent single 18-kip axle load (ESAL). The AASHTO method takes into account axle loads, axle configuration, and number of applications of the loads. The actual loading is related to ESAL by equivalence factors based on the terminal serviceability index p_t (Art. 16.22.4) and a parameter called structural number *SN*. The structural number is used to describe the overall thickness of the pavement (Art. 16.22.10).

Tables 16.6 and 16.7 list axle-load equivalence factors for single and tandem axles acting on flexible pavements with a p_t of 2.0 and 2.5, respectively. These tables can be used to convert mixed traffic loads to an equivalent number of 18-kip loads.

The accuracy of traffic estimates depends greatly on the accuracy of the following: load equivalence values, estimates of traffic volume and weight, prediction of ESAL over the design period, and interaction of age and traffic as it affects changes in PSI (Art. 16.22.4).

Traffic predictions are made for a convenient period of time, typically 20 years. Any period, however, may be used with the AASHTO design method since traffic is expressed as daily or total ESAL applications. The total ESAL applications are the number of repetitions of the loading that the pavement is expected to carry, from opening of the road to the time when it reaches its terminal value, for example, when $p_t = 2.0$.

For design purposes, the traffic must be distributed by direction and by lanes. Directional distribution is generally made by assigning 50% of the traffic to each direction (if special conditions do not warrant some other distribution). Lane distribution is usually made by assigning 100% of the traffic in each direction to each lane. Some states, however, have developed lane distribution percentages for highways with more than one lane in a given direction. Depending on the total number of lanes present, these percentages typically range from 60 to 100% of the one-directional traffic.

Because of the importance of the traffic data design of a pavement, the design team should work closely with the personnel involved in the gathering of this information. Poor traffic estimates can adversely affect highway performance and economy.

16.22.6 Subgrade Support for Flexible Pavements

A pavement is designed to distribute traffic loads to the subgrade, which must be capable of withstanding the resulting stresses. Hence the performance of the pavement depends greatly on the physical properties and condition of the subgrade soils. AASHTO characterizes the soil by its resilient modulus M_R, psi. The resilient modulus takes into account various nonlinear properties of the soil. (M_R replaces the soil support value *S* used in the past. The change was made because of the applicability of the resilient modulus to multilayered sys-

Table 16.6 Axle-Load Equivalence Factors for Flexible Pavement, $p_t = 2.0$*

Single Axles

Axle-Load, kips	Structural number *SN*					
	1	2	3	4	5	6
2	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002
4	0.002	0.003	0.002	0.002	0.002	0.002
6	0.01	0.01	0.01	0.01	0.01	0.01
8	0.03	0.04	0.04	0.03	0.03	0.03
10	0.08	0.08	0.09	0.08	0.08	0.08
12	0.16	0.18	0.19	0.18	0.17	0.17
14	0.32	0.34	0.35	0.35	0.34	0.33
16	0.59	0.60	0.61	0.61	0.60	0.60
18	1.00	1.00	1.00	1.00	1.00	1.00
20	1.61	1.59	1.56	1.55	1.57	1.60
22	2.49	2.44	2.35	2.31	2.35	2.41
24	3.71	3.62	3.43	3.33	3.40	3.51
26	5.36	5.21	4.88	4.68	4.77	4.96
28	7.54	7.31	6.78	6.42	6.52	6.83
30	10.38	10.03	9.24	8.65	8.73	9.17
32	14.00	13.51	12.37	11.46	11.48	12.17
34	18.55	17.87	16.30	14.97	14.87	15.63
36	24.20	23.30	21.16	19.28	19.02	19.93
38	31.14	29.95	27.12	24.55	24.03	25.10
40	39.57	38.02	34.34	30.92	30.04	31.25

Tandem Axles

Axle-Load, kips	Structural number *SN*					
	1	2	3	4	5	8
10	0.01	0.01	0.01	0.01	0.01	0.01
12	0.01	0.02	0.02	0.01	0.01	0.01
14	0.02	0.03	0.03	0.03	0.02	0.02
16	0.04	0.05	0.05	0.05	0.04	0.04
18	0.07	0.08	0.08	0.08	0.07	0.07
20	0.10	0.12	0.12	0.12	0.11	0.10
22	0.16	0.17	0.18	0.17	0.16	0.16
24	0.23	0.24	0.26	0.25	0.24	0.23
26	0.32	0.34	0.36	0.35	0.34	0.33
28	0.45	0.46	0.49	0.48	0.47	0.46
30	0.61	0.62	0.65	0.64	0.63	0.62
32	0.81	0.82	0.84	0.84	0.83	0.82
34	1.06	1.07	1.08	1.08	1.08	1.07
36	1.38	1.38	1.38	1.38	1.38	1.38
38	1.76	1.75	1.73	1.72	1.73	1.74
40	2.22	2.19	2.15	2.13	2.16	2.18
42	2.77	2.73	2.64	2.62	2.66	2.70
44	3.42	3.36	3.23	3.18	3.24	3.31
46	4.20	4.11	3.92	3.83	3.91	4.02
48	5.10	4.98	4.72	4.58	4.68	4.83

*From "Guide for Design of Pavement Structures," American Association of State Highway and Transportation Officials.

Table 16.7 Axle-Load Equivalence Factors for Flexible Pavement, p_t = 2.5*

Single Axles

Axle-Load, kips	Structural number SN					
	1	2	3	4	5	6
2	0.0004	0.0004	0.0003	0.0002	0.0002	0.0002
4	0.003	0.004	0.004	0.003	0.003	0.002
6	0.01	0.02	0.02	0.01	0.01	0.01
8	0.03	0.05	0.05	0.04	0.03	0.03
10	0.08	0.10	0.12	0.10	0.09	0.08
12	0.17	0.20	0.23	0.21	0.19	0.18
14	0.33	0.36	0.40	0.39	0.36	0.34
16	0.59	0.61	0.65	0.65	0.62	0.61
18	1.00	1.00	1.00	1.00	1.00	1.00
20	2.61	1.57	1.49	1.47	1.51	1.55
22	2.48	2.38	2.17	2.09	2.18	2.30
24	3.69	3.49	3.09	2.89	3.03	3.27
26	5.33	4.99	4.31	3.91	4.09	4.48
28	7.49	6.98	5.90	5.21	5.39	5.98
30	10.31	9.55	7.94	6.83	6.97	7.79
32	13.90	12.82	10.52	8.85	8.88	9.95
34	18.41	16.94	13.74	11.34	11.18	12.51
36	24.02	22.04	17.73	14.38	13.93	15.50
38	30.90	28.30	22.61	18.06	17.20	18.98
40	39.26	35.89	28.51	22.50	21.08	23.04

Tandem Axles

Axle-Load, kips	Structural number SN					
	1	2	3	4	5	6
10	0.01	0.01	0.01	0.01	0.01	0.01
12	0.02	0.02	0.02	0.02	0.01	0.01
14	0.03	0.04	0.04	0.03	0.03	0.02
16	0.04	0.07	0.07	0.06	0.05	0.04
18	0.07	0.10	0.11	0.09	0.08	0.07
20	0.11	0.14	0.16	0.14	0.12	0.11
22	0.16	0.20	0.23	0.21	0.18	0.17
24	0.23	0.27	0.31	0.29	0.26	0.24
26	0.33	0.37	0.42	0.40	0.36	0.34
28	0.45	0.49	0.55	0.53	0.50	0.47
30	0.61	0.65	0.70	0.70	0.66	0.63
32	0.81	0.84	0.89	0.89	0.86	0.83
34	1.06	1.08	1.11	1.11	1.09	1.08
36	1.38	1.38	1.38	1.38	1.38	1.38
38	1.75	1.73	1.69	1.68	1.70	1.73
40	2.21	2.16	2.06	2.03	2.08	2.14
42	2.76	2.67	2.49	2.43	2.51	2.61
44	3.41	3.27	2.99	2.88	3.00	3.16
46	4.18	3.98	3.58	3.40	3.55	3.79
48	5.08	4.80	4.25	3.98	4.17	4.49

*From "Guide for Design of Pavement Structures," American Association of State Highway and Transportation Officials.

tems in general and pavement structures in particular.) Because some transportation agencies do not have the ability to perform the resilient modulus test (described in AASHTO Test Method T274), the AASHTO "Guide for Design of Pavement Structures" contains correlations that relate the frequently used California bearing ratios (CBR) and stabilometer R values to an equivalent M_R.

An equivalent M_R can be determined for the Corps of Engineers CBR value from

$$M_R = 1500\,\mathrm{CBR} \tag{16.17}$$

Equation (16.17) is valid for fine-grain soils with a soaked CBR of 10 or less. An equivalent value M_R based on an R value can be determined for fine-grain soils with an R value less than or equal to 20 from

$$M_R = 1000 + 555R \tag{16.18}$$

The AASHTO "Guide" contains design curves for conversion to a structural number SN (Art. 16.22.9).

The resilient modulus is based on the properties of the compacted subgrade soils. It may be necessary, however, to include the properties of in situ materials in the uncompacted foundation if these materials are especially weak. Also, compaction of the subgrade is essential to ensure adequate performance and reliability.

16.22.7 Flexible-Pavement Material

For flexible pavements, materials used for subbase, base, and surface courses differ. Article 16.22.1 describes the properties and characteristics of these layers. For more detailed information, see AASHTO "Guide for the Design of Pavement Structures" and "Construction Manual for Highway Construction."

In addition to the aforementioned three principal layers, the prepared roadbed is an important component of a flexible pavement, and a drainage layer also may be necessary. The prepared roadbed may be a layer of compacted roadbed soil or select borrow material that is compacted to a specified density. Examples of a drainage layer are given in Fig. 16.21. Figure 16.21*a* shows a base course that serves also as a drainage layer, whereas Fig. 16.21*b* shows a drainage layer between the subbase and the subgrade.

16.22.8 Flexible-Pavement Drainage

Rainfall is one of the principal environmental conditions that affects the design and performance of pavements. The major concern with rainwater is that it may penetrate through the pavement into the roadbed soil and weaken it. Proper drainage is an important element in preventing this. Experience has shown that pavements that are not properly drained deteriorate prematurely, especially when exposed to heavy traffic volumes and their related loads.

Articles 16.16 and 16.17 discuss the adverse effects when water penetrates a pavement and describes some methods employed to prevent this and to remove water from the surface of the roadway. The AASHTO design method for flexible pavements takes into account the impact of swelling, frost heave, and moisture on roadbed soil and base strength. This is done by multiplying the structural layer coefficients a_1 and a_2 (Art. 16.22.9) by a factor m_i that takes into account the quality of drainage and the percent of time the pavement is subjected to moisture levels approaching saturation. The quality of drainage is indicated by the amount of time needed to drain the base layer to 50% of saturation.

("Guide for Design of Pavement Structures," American Association of State Highway and Transportation Officials.)

16.22.9 Structural Numbers for Flexible Pavements

The design of a flexible pavement or surface treatment expected to carry more than 50,000 repetitions of ESAL (Art. 16.22.5) requires identification of a structural number SN that is used as a measure of the ability of the pavement to withstand anticipated axle loads. In the AASHTO design method, the structural number is defined by

$$SN = SN_1 + SN_2 + SN_3 \tag{16.19}$$

where SN_1 = structural number for the surface course = a_1D_1

a_1 = layer coefficient for the surface course

D_1 = actual thickness of the surface course, in

SN_2 = structural number for the base course = $a_2D_2m_2$
a_2 = layer coefficient for the base course
D_2 = actual thickness of the base course, in
m_2 = drainage coefficient for the base course
SN_3 = structural number for the subbase course = $a_3D_3m_3$
a_3 = layer coefficient for the subbase course
D_3 = actual thickness of the subbase course, in
m_3 = drainage coefficient for the subbase

The layer coefficients a_n are assigned to materials used in each layer to convert structural numbers to actual thickness. They are a measure of the relative ability of the materials to function as a structural component of the pavement. Many transportation agencies have their own values for these coefficients. As a guide, the layer coefficients may be 0.44 for asphaltic-concrete surface course, 0.14 for crushed-stone base course, and 0.11 for sandy-gravel subbase course. The drainage coefficient m_n is discussed in Art. 16.22.8.

The thicknesses D_1, D_2, and D_3 should be rounded to the nearest ½ in. Selection of layer thicknesses usually is based on agency standards, maintainability of the pavement, and economic feasibility.

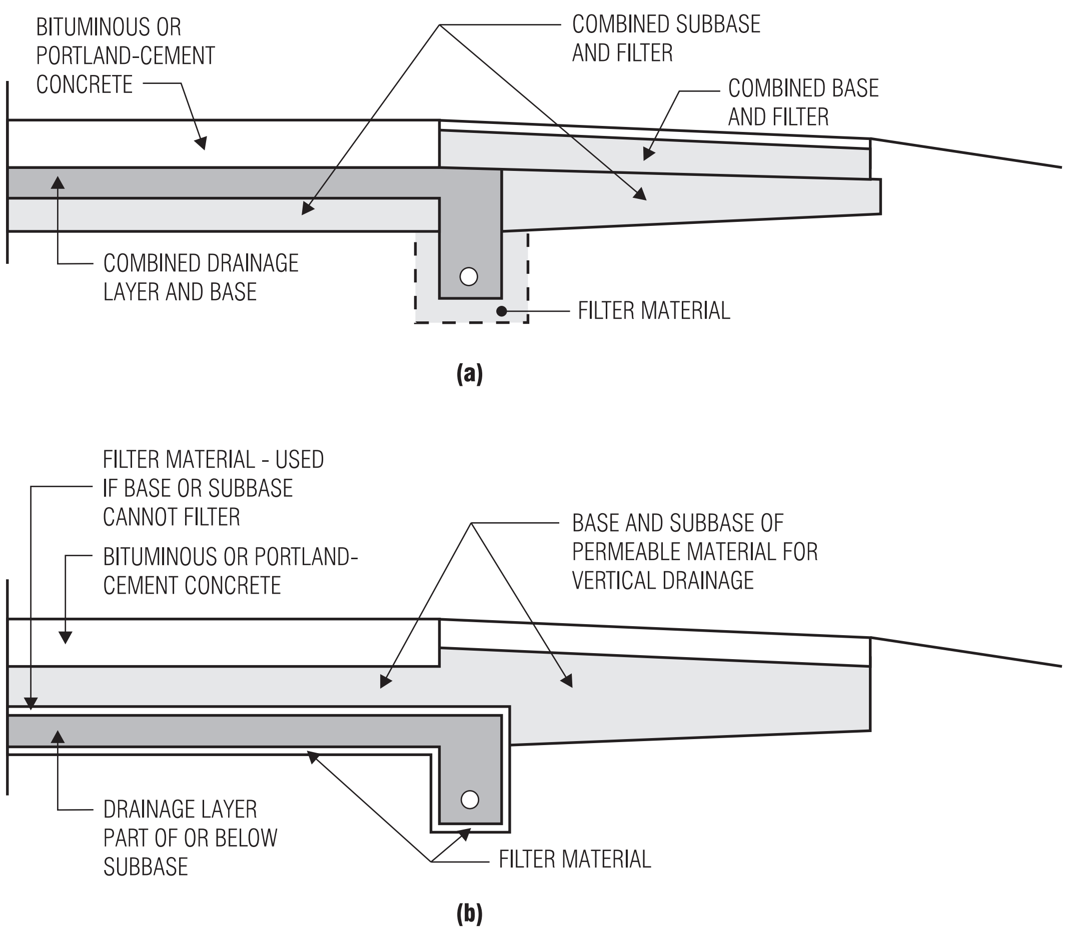

Fig. 16.21 Drainage layers under pavements: (*a*) base used as the drainage layer; (*b*) drainage layer as part of or below the subbase.

See also Art. 16.22.10 and the AASHTO "Guide for Design of Pavement Structures."

16.22.10 Determination of Course Thicknesses

The thickness to be used for the various layers of a flexible pavement is, along with other parameters, a function of the material used and the load the pavement is expected to withstand. Minimum thickness for each layer depends on the size of aggregate used. With aggregate size as the controlling criterion, the following are minimum layer thicknesses: surface course, 1½ in; base course, 3 in; and subbase course, 4 in. Table 16.8 lists minimum thicknesses recommended by AASHTO for various levels of ESAL. These are practical thicknesses and vary with local conditions and design practices.

A flexible pavement is essentially a composite of layers (Fig. 16.22) and is designed as such. The first step is to determine the structural number *SN* needed for the combination of layers above the subgrade with the use of the resilient modulus (see Art. 16.22.6). Next, the structural numbers needed for the combination of layers above the subbase and for the surface course are calculated. Taking into account the differences between these calculated structural numbers, a maximum allowable thickness for any layer can be found. Therefore, to determine the maximum allowable structural number for the subbase material, subtract the structural number required for the layers above the subbase from the structural number required for the subgrade. Repeat this process for the other layers in the pavement. After the structural numbers have been determined, the respective layer thicknesses can be calculated as follows:

Table 16.8 Minimum Layer Thickness, in, Based on ESAL*

Traffic, ESAL	Asphalt concrete, in	Aggregate base, in
Less than 50,000	1.0†	4
50,000 – 150,000	2.0	4
150,001 – 500,000	2.5	4
500,001 – 2,000,000	3.0	6
2,000,001 – 7,000,000	3.5	6
Greater than 7,000,000	4.0	6

*Adapted from AASHTO "Guide for Design of Pavement Structures."
† For surface treatment.

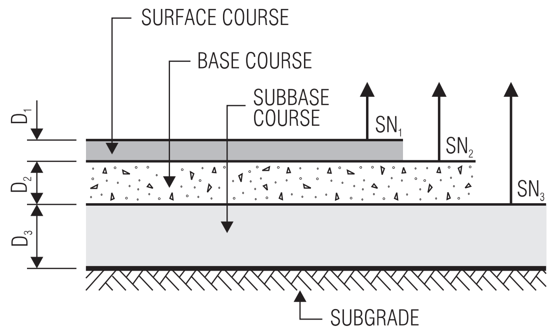

Fig. 16.22 Composite of layers forming a flexible pavement. *SN* indicates structural number of a layer.

1. The thickness D_1 of the surface course is determined by dividing the structural number required SN_1 for the surface course by the layer coefficient a_1. Select a thickness D'_1 by rounding the calculated value to the nearest larger ½ in or a more practical dimension.
2. The structural number supplied then is $SN'_1 = a_1D'_1$, which is larger than SN_1.
3. The thickness D_2 to be used for the base course should be chosen selected equal to or larger than $(SN_2 - SN'_1)/a_2m_2$, where SN_2 is the required structural number for the base and surface layers. The sum of the structural numbers supplied for the base and surface courses then should be equal to or larger than SN_2.
4. The thickness D_3 to be used for the subbase course should be selected equal to or larger than $[SN_3 - (SN'_1 + SN'_2)]/a_3m_3$.

The AASHTO "Guide" presents various charts and design aids for determining the structural numbers layer thicknesses required for a pavement. A limiting criterion of this method is that it cannot be used to determine the *SN* required above subbase or base materials possessing an elastic modulus greater than 40,000 psi. In such instances, the thickness of a layer above the high-modulus layer should be based on economic and practical minimum-thickness considerations.

See also "Thickness Design—Full Depth Asphalt Pavement Structures for Highways and Streets," Manual MS-1, The Asphalt Institute, College Park, MD 10740.

16.23 Alternative Flexible Pavements

A variety of technologies are available as alternatives to the type discussed in Art. 16.22. Included in this category are porous pavements, sulfur-asphalt mixes, hydrated-lime additives, rubberized wearing surfaces, recycled asphalt pavements, and the Superpave mix design system.

16.23.1 Porous Pavements

These are essentially asphalt pavements without any fines (sand) in the mix. This type of pavement contains voids through which rainwater is allowed to seep into the subgrade. This characteristic offers several advantages: Removal of water from the pavement decreases the possibility of damage from trapped water, thus increasing pavement life. Also, if storm water from the pavement can percolate into the soil, a smaller highway drainage system is needed. If there is an existing storm sewer, the risk of overloading it is also greatly reduced. Furthermore, porous pavements enhance traffic safety by decreasing the risk of hydroplaning (wet skidding). In addition, driver visibility of pavement markings does not suffer in rain because the water percolates rapidly through the porous asphalt surface. From an aesthetic viewpoint, porous pavement is not objectionable since there is no basic visual difference between porous and conventional, nonpermeable pavements.

Porous pavements are generally used in highways, local streets, and parking lots. For parking lots, porous pavements are advantageous because rain seeping from them into the subgrade promotes healthy growth of trees, shrubs, ground cover, and other plantings and thus makes a parking area and associated landscaping more aesthetically pleasing. For a parking area, a typical porous pavement consists of a $2\frac{1}{2}$-in surface course of porous bituminous concrete over a 12-in graded, crushed-stone base. The base course is layered. Small stones form the top layer so that a paving machine can create a smooth surface for application of the surface course.

16.23.2 Sulfur-Asphalt Mixes

Sulfur is used in bituminous pavements in several ways. In one method, sulfur serves as a filler. It is added to a hot sand-asphalt mix after the asphalt and aggregate have been mixed. The sulfur fills the voids and locks the sand particles, stabilizing the mix. In another method, sulfur and asphalt are blended to form sulfur-extended asphalt (SEA). The hot sulfur is dispersed into the asphalt to create a binder that is then mixed with the aggregate. Production of SEA requires only a slight modification of the hot-mix plant. Otherwise, the construction operations and equipment for SEA are the same as that for asphalt concrete.

Sulfur is also used for roads in areas subject to permafrost. Conventional highway construction practice calls for gravel depths of 5 ft or more below the ground surface to provide a stable load-bearing surface. Also, thermal insulation is installed below the gravel to protect the underlying permafrost. When gravel is not available locally, as is often the case in many northern areas, it must be transported to the project site from elsewhere at considerable expense. Construction costs can be cut, however, by reducing significantly the amount of gravel required through use of sulfur in foam form with gravel. One test showed that 7 ft of gravel could be replaced by only 3 ft of gravel set atop 3 to 4 in of sulfur foam.

There are, however, health hazards associated with the use of sulfur in general. For instance, noxious gases, such as sulfur dioxide and hydrogen disulfide, can be generated at the plant and the construction site.

16.23.3 Hydrated Lime

This is widely used in hot mixes that contain marginally acceptable aggregates. The lime acts as a chemical additive rather than a void filler. It increases the strength and stability of an asphalt mix while making it more water resistant. Also by hardening mixes, it allows faster compaction and yields higher densities.

16.23.4 Rubber in Wearing Courses

Rubber is used to improve the paving qualities of hot mixes used in bituminous wearing courses. For this purpose, rubber may be added to an asphalt-concrete mix or applied to the pavement surface after placement and compaction. The rubber reduces temperature susceptibility, decreases raveling, offers better void control, and lessens the tendency to flow, improving flexibility and adhesion to aggregates.

16.23.5 Superpave Mix Design System

Developed by the Strategic Highway Research Program (SHRP), the Superpave mix design system is a method of designing flexible-pavement mixes that are tailored to specific project characteristics. These include traffic, environment, pavement structural section, and reliability.

The Superpave mix design system assists in selection of combinations of asphalt binder, aggregate, and any necessary modifiers to obtain a desired level of pavement performance. The goal of the system is to create an *ideal* blend of asphalt binder and aggregate for production of the lowest-cost pavement for the anticipated level of service.

The Superpave system applies to three different levels of traffic, low, intermediate, and high, and employs laboratory and field testing techniques. Computer software based on the Superpave specifications is available to assist in the process. The software and associated specifications perform analysis and design of multiple layer pavements consisting of base, binder, and surface courses. For example, selection of the necessary materials used in the Superpave mix is based on, among other things, the design ESAL for the project (Art. 16.22.5). ESALs are used to ascertain whether the anticipated traffic level is low, intermediate, or high. Also taken into account are pavement environmental conditions influenced by climate. Based on these conditions, an asphalt binder, for example, can be chosen.

The Superpave system also allows for addition of modifiers, such as fibers or hydrated lime, to the mix to enhance the ability of the paving mixes to avoid pavement distresses. While the system does not offer a list of modifiers for correction of specific pavement distress, it does offer a guide based on AASHTO Practice PP5, "The Laboratory Evaluation of Modified Asphalt Systems," to assist in the selection of appropriate modifiers to enhance the performance of the pavement.

("The Superpave Mix Design Manual for New Construction and Overlays," SHRP-A-407, Strategic Highway Research Program, National Research Council, 2101 Constitution Ave., NW, Washington, DC 20418.)

16.23.6 Recycling of Asphalt Pavements

The materials in an asphalt pavement that is scheduled for replacement can be reused as ingredients of a new surface course or a new pavement, including underlying, untreated base material. The recycling may be performed in place or at a central plant.

When the asphalt pavement is to be recycled at the site, a process known as in-place, cold-mix, asphalt-pavement recycling is used. In this process, the existing pavement materials are ripped, broken, pulverized, and mixed in place with asphalt or other materials, such as aggregates or stabilizing agents. The other materials usually are required to provide a higher-strength base. The process requires that an asphalt surface course be placed on top of the recycled layer. One drawback to this process is that quality control is not so good in the field as it would be at a central plant. Another is that maintenance of traffic is difficult because of the necessity of avoiding interference with the recycling equipment. The cold-mix process, however, can also be conducted at a central plant where enhanced quality control provides higher mix efficiency and reliability. Central plants also offer higher production capacity and better uniformity and reliability.

An alternative to the cold-mix process is hot-mix, asphalt-pavement recycling, which is performed at a central plant. In this process, reclaimed asphalt-pavement materials are removed from an existing roadway in a fashion similar to that described for the cold-mix process and combined in the central plant with new asphalt or recycled agents. The hot-mix method also sometime utilizes uncoated aggregates from the base to produce the hot mix. For the hot-mix recycling process, one of the following types of plants is generally used: batch, drum mixer, or continuous mixer.

Several factors affect the feasibility of a recycling project. These include availability of recycling equipment, impact on traffic through the construction site, and the size and location of the project. In the right situation, however, recycling can offer many economic and environmental advantages.

16.24 Rigid Pavements

A rigid pavement typically consists of a portland cement–concrete slab resting on a subbase course. (The subbase course may be omitted when the subgrade material is granular.) The slab possesses beamlike characteristics that allow it to span across irregularities in the underlying material. When designed and constructed properly, rigid pavements provide many years of service with relatively low maintenance.

16.24.1 Subbase for a Rigid Pavement

This consists of one or more compacted layers of granular or stabilized material placed between the subgrade and the rigid slab. The subbase provides a uniform, stable, and permanent support for the concrete slab. It also can increase the modulus of subgrade reaction k, reduce or avert the adverse effects of frost, provide a working platform for equipment during construction, and prevent pumping of fine-grain soils at joints, cracks, and edges of the rigid slab.

In design and maintenance of a rigid pavement, a major concern is prevention of accumulation of water on or in the subbase or roadbed soils. AASHTO recommends that, if needed for drainage purposes, the subbase layer be carried 1 to 3 ft beyond the paved roadway width or to the inslope. Another concern is prevention of erosion, particularly at slab joints and pavement edges. To compensate for this, lean concrete or porous layers are sometimes used as the subbase material. This practice, however, requires close inspection by design and maintenance personnel.

16.24.2 Types of Concrete Pavements

A concrete pavement may be plain concrete, reinforced concrete, or prestressed concrete. Figure 16.23 shows a cross section of a reinforced concrete pavement. The half cross section in Fig. 16.23*a* is shown reinforced whereas that in Fig. 16.23*b* is unreinforced.

Reinforced concrete pavements may be jointed or continuously reinforced. Continuously reinforced pavements eliminate the need for transverse joints but do require construction joints or joints at physical interruptions of the highway, such as bridges. Plain-concrete pavements have no reinforcement except for steel tie bars used to hold longitudinal joints tightly closed.

Jointed Reinforced Concrete Pavement ■ The main function of reinforcing steel in a jointed concrete pavement is to control cracking caused by thermal expansion and contraction, soil movement, and moisture. The amount and spacing of transverse and longitudinal reinforcing steel required for this purpose depend on slab length, type of steel used, and resistance between the bottom of the slab and the top of the underlying subgrade (or subbase) layer.

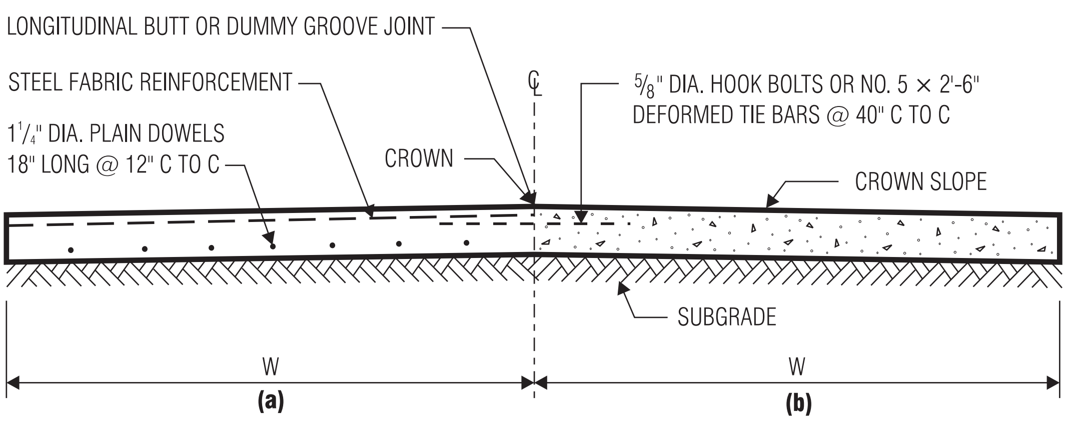

Fig. 16.23 Components of a reinforced concrete pavement.

Continuously Reinforced Concrete Pavement ■ The principal reinforcement in a continuously reinforced pavement is longitudinal steel, which may be reinforcing bars or deformed wire fabric. It is used to control cracking caused by volume changes in the concrete.

In addition to the longitudinal steel, transverse reinforcement may be provided to control the width of longitudinal cracks. When longitudinal cracking is not expected to be troublesome, transverse reinforcement may not be required.

Design of continuously reinforced pavements should take into account the properties of the concrete used. Specifically, the concrete properties that affect the design of continuously reinforced pavements are tensile strength, shrinkage characteristics, and thermal coefficient. Pavement design should also take into account the anticipated drop in temperature, which for design purposes is the difference between the average concrete curing temperature and a design minimum temperature.

Longitudinal reinforcing steel used typically consists of No. 5 and No. 6 deformed bars. When wire fabric is used, the diameter should be of sufficient size that corrosion and deterioration will not significantly impair the cross-sectional properties of the fabric.

16.24.3 Reinforced Concrete Pavement Slabs

These are typically constructed of portland cement–concrete, reinforcing steel, load-transfer devices, and joint sealing materials. These materials should conform to the appropriate AASHTO or agency specifications to ensure that pavement distortion or disintegration is minimized.

The thickness of concrete pavement slabs generally is determined with the use of design charts or computer software. For details of a design method, see "Guide for Design of Pavement Structures," American Association of State Highway and Transportation Officials (AASHTO). In this method, the effects of traffic loads are determined by the use of an equivalent single 18-kip axle load (ESAL). See Arts. 16.22.4 and 16.22.5.

Concrete pavement slabs may be subject to surface deterioration caused by deicing agents or expansion and contraction due to temperature changes. To combat such forms of deterioration, air-entrained concrete is used. Air entrainment also improves the workability of the concrete mix. The design of the mix and its material specifications should be in accordance with the AASHTO "Guide Specifications for Highway Construction" and AASHTO "Standard Specifications for Transportation Materials." Specifications for portland cement concrete are also promulgated by ASTM. The AASHTO and ASTM specifications contain requirements for the properties of the cement, coarse aggregates, and fine aggregates to be used in the mix.

16.24.4 Reinforcing Steel for Concrete Pavement

The purpose of reinforcing steel in a concrete pavement slab is to control cracking as well as tie together slab segments and act as a load-transfer mechanism at joints. The reinforcing steel, whether bars or wire fabric, is generally deformed; that is, the steel surfaces have a ribbed deformation that enhances bond between steel and concrete.

The reinforcing steel that is used primarily to control cracking is known as **temperature steel**. The steel used to tie two slabs together is known as **tie bars**. The steel bars that act as a load-transfer mechanism are called **dowels**.

Temperature steel may consist of deformed bars, a bar mat, or a wire mesh. The purpose of temperature steel is to control the width of cracks, not necessarily to prevent cracking. If a smooth wire mesh is used, the bond between steel and concrete is developed through the welded cross wires. When a deformed wire fabric is used, the bond is developed through the deformations on the steel and at the welded intersections. The steel mesh should be given adequate concrete cover on top, usually about 3 in. The amount of steel to be provided depends on slab thickness, length, and material properties, such as type of concrete and reinforcing steel used.

16.24.5 Tie Bars

These are installed between abutting slabs to tie them together. For this purpose, the tie bars, which may be connectors or deformed bars, should have sufficient tensile strength to prevent the slabs from separating. (Tie bars are not intended to serve as load-transfer devices.) The tendency for the slabs to separate arises because they try to shorten when the temperature drops or moisture content of the concrete decreases (as is the case when the concrete cures). The resistance to movement provided by the tie bars induces tensile stresses in the concrete, which must have sufficient tensile strength to withstand these stresses, or reinforcing steel should be provided for this purpose. To facilitate bonding, the tie bars are usually equipped with a hook. It is often advantageous to provide the bars with a protective coating, especially when the pavement slab is exposed to deicing agents.

16.24.6 Load-Transfer Devices

Load-transfer devices are installed between the ends of abutting slabs to transfer traffic loads from one to the other yet offer little or no resistance to longitudinal movements of the slabs. The most common form of load-transfer device is a large-diameter dowel. Other mechanical devices, however, have been used successfully. It is also possible to achieve load transfer with the interlocking of aggregate alone.

A dowel provides flexural, shear, and bearing resistance. One end of the dowel is bonded to the concrete. The other end is allowed to move freely. For this purpose, half the dowel adjoining this end may be greased, painted, or coated with asphalt, thus preventing the dowel from bonding to the concrete. As a result, the dowel can slide freely in the concrete after embedment in the end of a slab. To ensure proper movement of the dowels, it is essential to maintain alignment between the abutting slabs.

Although offering little restraint to longitudinal movement of the slabs, load-transfer devices should also be mechanically stable under wheel loads and cyclical loading. It is often beneficial to provide the dowel, or other device, with a protective coating when the slab may be exposed to corrosive elements.

Dowel size depends on slab thickness and other site-specific criteria. A general rule of thumb is that the dowel diameter should be equal to one-eighth the slab thickness; for example, for a 9-in slab, a diameter of $1\frac{1}{8}$ in might be used. Dowel spacing is generally 12 in and dowel length 18 in.

16.24.7 Joints in Concrete Pavement

Joints are formed in concrete pavement to reduce the effects of expansion and contraction, facilitate concrete placement, and allow for bonding of abutting slabs. Joints may be perpendicular to the pavement centerline (transverse) and, depending on the intended function of the joints, longitudinal.

Transverse Expansion Joints ■ The principal function of an expansion joint in concrete pavement is to allow movement of the slab due to changes in temperature. For example, when temperature increases, a slab increases in length, thereby creating compressive stresses in the concrete. If expansion joints are not provided, the slab, depending on its length, might buckle upward or blow up.

In concrete pavement, expansion joints are generally placed every 40 to 60 ft along the length of the pavement. The joints, which may range in thickness from $\frac{3}{4}$ to 1 in, should incorporate appropriate load-transfer devices (Art. 16.22.6). Fillers, such as rubber, bitumen, or cork, that permit expansion of the slab and exclude dirt should be placed in the joints.

Some transportation agencies specify expansion joints, but others do not and instead employ alternate methods to minimize the potential for blowups. One way this is done is to use cement and aggregates with properties that limit the amount that slabs increase in length with increases in temperature.

Transverse Contraction Joints ■ Contraction joints are provided to limit the effects of tensile forces in a concrete slab caused by a drop in temperature. The objective is to weaken the slab so that if the tensile forces are large enough to crack it, the cracks will form at the joints. The depth of contraction joints generally is only one-quarter the thickness of the slab. When properly designed and spaced, however, they can also minimize cracking of the slab outside the joints.

Contraction joints can be formed by sawing into the hardened concrete, by inserting plastic inserts at joint locations before concrete is cast, or by tooling the concrete after placement but before the concrete has fully hardened. When aggregate interlocking is insufficient for transferring load between the pavement sections on either side of a joint, an appropriate load-transfer mechanism should be incorporated in the joint.

Longitudinal Joints ■ These are formed parallel to the highway centerline to facilitate lane construction and prevent propagation of irregular, longitudinal cracks. The joints can be keyed, butted, mechanically formed, or saw grooved. To prevent adjacent lanes from separating or faulting, steel tie bars or connections should be embedded in the concrete, perpendicular to the joints.

Longitudinal joints are formed or sawed to a minimum depth of one-fourth the slab thickness. The maximum longitudinal joint spacing recommended by AASHTO is 16 ft.

Construction Joints ■ When concrete placement for a concrete slab is interrupted, a construction joint is desirable at the *cold joint* between the two slab sections. In preparation for the interruption, a vertical face is formed with a header board at the end of the slab being cast. The header board is equipped with a protrusion that, when formed into the concrete, creates a key way for load transfer between the adjoining slab sections. It is also sometimes desirable to use deformed tie bars to hold the joint closed.

Joint Sealing ■ Many highway agencies specify that all joints be cleaned and also sealed to exclude dirt and water. Others seal only expansion joints. The basic types of sealants used are liquid seals, preformed elastomeric seals, and cork expansion-joint filler.

Liquid seals are poured into a joint where they are allowed to set. Types of liquid seals used include asphalt, hot-poured rubber, elastomeric compounds, silicone, and polymers.

Preformed elastomeric seals consists of extruded neoprene strips with internal webs that exert an outward force against the faces of the joint. The type of elastomeric seal to specify depends on the anticipated slab movement at the joint. The seals are provided with a coat of adhesive to fasten them to the faces of the joint.

Highway Intersections and Interchanges

An intersection is the junction or crossing of two or more roads at the same or different elevations. When the roads are at the same level, the intersection is called an at-grade intersection. When the roads are at different elevations, the intersection is referred to as a grade separation when there is no connection between the intersecting roads or as an interchange when connecting roads, such as ramps or turning roadways, permit movement of vehicles between the intersecting roads.

Intersections should be kept simple so that necessary movements are obvious to drivers. Uniformity of intersections is important to avoid driver confusion. Factors to be considered for this purpose include design speed, intersection angles (90° is preferred), intersection curves, vehicle turning paths, roadway widths, and traffic control devices.

16.25 At-Grade Intersections

The junction or crossing of two or more highways at a point of common elevation is called an at-grade intersection.

Intersections of highways and railways at grade should be provided with protective and warning devices. Sight distance is an important design consideration when only advance warning of approaching trains and railway crossbuck signs are installed.

16.25.1 Geometric Design of At-Grade Intersections

Major influences on the geometric design of at-grade intersections include human factors, traffic considerations, physical elements, and economic factors. The goal is to reduce or eliminate the potential for accidents involving vehicular, bicycle, or pedestrian traffic through the intersection. Also, natural transitional paths for traffic must be provided.

Human Factors ■ Design of an intersection is affected by human factors, such as driving habits, the ability of drivers to make decisions, adequate advance warning to drivers regarding the presence of an intersection, driver decision and reaction time, and the presence of pedestrians at the intersection.

Traffic Considerations ■ Traffic volume and movement impact the design of an at-grade intersection. Both the design and actual capacity of the intersecting highways should be taken into account. Also of concern are the design-hour turning movement and other movements, such as diverging, merging, weaving, and crossing.

Other traffic criteria include vehicle size, speed and operating characteristics, transit involvement, and, if applicable, the history of accidents at the site.

Storage requirements for traffic-signal-controlled approaches should also be taken into account.

Physical Elements ■ Geometric and site-specific features whether natural such as topography and vegetation, or man-made, such as signs, have important influences on design of at-grade intersections. The horizontal and vertical alignment of the intersecting roadways also greatly affects the design. Both of these elements impact sight distance and angle of intersection. Other features affecting the design are traffic control devices, lighting equipment, and safety appurtenances.

Preexisting site conditions, including abutting property uses, such as shopping areas and industrial complexes, and the presence of sidewalks and their associated pedestrian traffic, should also be factored into the design process.

Economic Factors ■ Design of an at-grade intersection should be both practicable and economically feasible. The cost of required improvements along with the impact on abutting residential or commercial properties should be taken into account.

16.25.2 Types of At-Grade Intersections

Each highway that radiates from an intersection and forms part of it is known as an **intersection leg**. The intersection of two highways generally results in four legs. Intersections with more than four legs are not recommended.

Three-Leg Intersections ■ A three-leg intersection is formed when one highway starts or terminates at a junction with another highway (Fig. 16.24). Unchannelized T intersections (Fig. 16.24*a*) are usually employed at the intersection of minor roads with more important highways at an angle not exceeding 30° from the normal. At times, a right-turn lane is provided on one side of the through highway (Fig. 16 24*b*) This type of turn lane is used when right-turning traffic from the through highway is significant and left-turning traffic from the through highway is minor.

Four-Leg Intersections ■ A four-leg intersection is formed when two highways cross at grade (Fig. 16.25). The design of four-leg intersections follows many of the general guidelines for three-leg intersections. As with T intersections, the roadway intersection angle typically should not be more than 30° from the normal. Figure 16.25*c* shows a four-leg intersection of a through highway and a minor highway. The through highway is flared to provide additional capacity for through and turning movements. The flaring is provided through incorporation of parallel auxiliary lanes that are required for major highways requiring an uninterrupted flow capacity. Flaring may also be needed where cross traffic is sufficiently high to warrant signal control.

Channelization at Intersections ■ This is a method of creating defined paths for vehicle travel by installing traffic islands or pavement markings at at-grade intersections. These *defined paths* provide for the safe and orderly movement of both vehicles and pedestrians through the intersections. Channelized intersections are illustrated in Figs. 16.24 and 16.25.

Channelization should be used prudently; excessive use of channelization may worsen rather than improve conditions at an intersection. When properly implemented, channelization can dramatically reduce accidents at at-grade intersections. Factors that influence design of a channelized intersection include type of design vehicle, vehicle speed, cross sections of roadways, anticipated volumes of vehicle and pedestrian traffic, locations of bus stops, and type and location of traffic-control devices.

Figure 16.24*c* and *d* shows examples of channelization of three-leg intersections. In these intersections, the return radius between the intersecting roadways is larger than that used for the unchannelized intersections in Fig. 16.24*a* and *b*. This is done to provide space for the channelizing traffic islands. Whether the approach roadway should have a turning lane for right-turning traffic depends on the volume of traffic that will make right turns. When speeds or turning paths are above a prescribed minimum, the incorporation of dual right-turning roadways, as shown in Fig. 16.24*d*, may be required.

Channelization of four-leg intersections is often incorporated for many of the reasons given above for three-leg intersections. In Fig. 16.25*b*, a four-leg intersection with right-turning lanes in all four quadrants is shown. This approach is taken when space is readily available and turning movements are critical. This type of channelized, four-leg intersection is used frequently in suburban areas where pedestrian traffic is present.

16.25.3 Horizontal and Vertical Alignment at Intersections

Alignment geometries play a critical role in the design of an at-grade intersection. In the vertical plane, it is important that the profiles of the intersecting roadways be as flat as possible (preferably less than 3% through the intersection). Also, the horizontal alignment should be as straight as practical. Grade and curvature have considerable impact on sight distance at intersections, where it is desirable to have sight distances greater than specified minimum values. Adverse sight-distance conditions can be the source of accidents, because of driver inability to see other vehicles or discern the messages of traffic-control devices.

Horizontal Alignment ■ A general rule of thumb when laying out the horizontal alignments of intersecting roadways is to minimize the deviation of the intersection angle from 90°. Excessively skewed intersections can result in poor driving conditions, especially for trucks. Where the acute angles are formed, visibility may be limited and the exposure time of vehicles dangerously large as they cross the main flow of traffic. Where the obtuse angles are formed, blind spots may occur on the right side of the vehicle. Therefore, it is desirable to have the intersection angle as close to 90° as possible.

Vertical Alignment ■ Maximizing driver sight distance should be the goal of the vertical alignment. Proper sight distance should be provided along each highway and across the corners.

Whenever possible, major grade changes should be avoided at intersections. Generally, the profile of

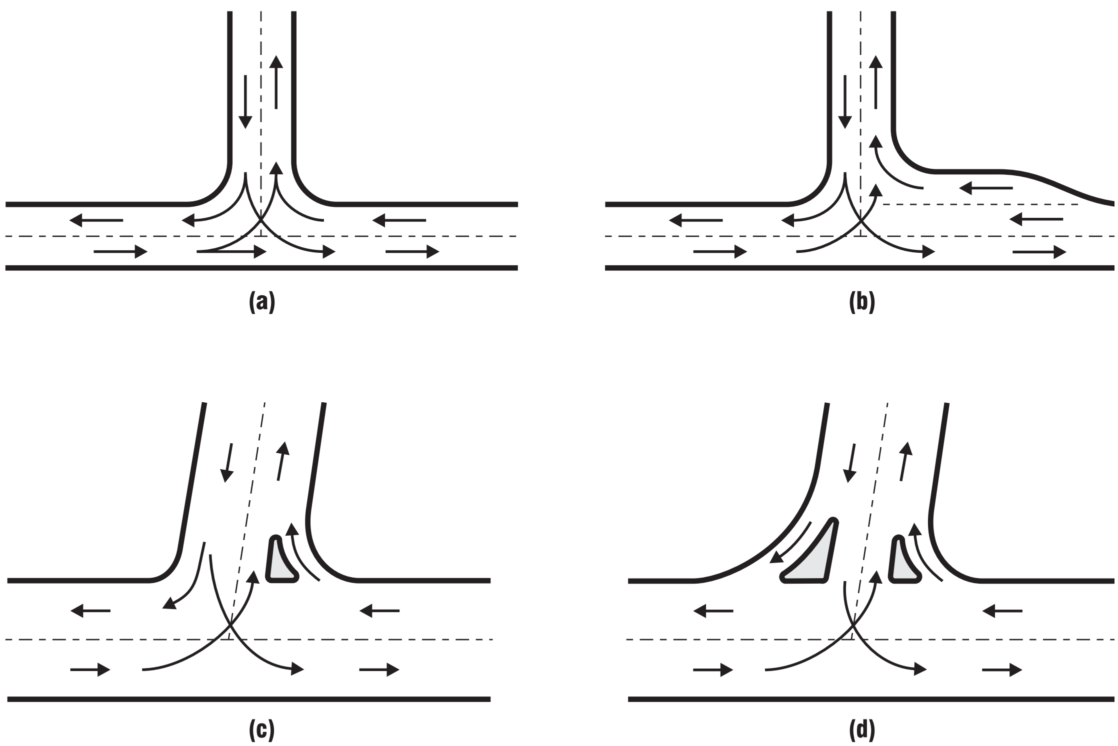

Fig. 16.24 Types of at-grade T (three-leg) intersections: (*a*) unchannelized; (*b*) intersection with a right-turn lane; (*c*) intersection with a single-turning roadway; (*d*) channelized intersection with a pair of turning roadways.

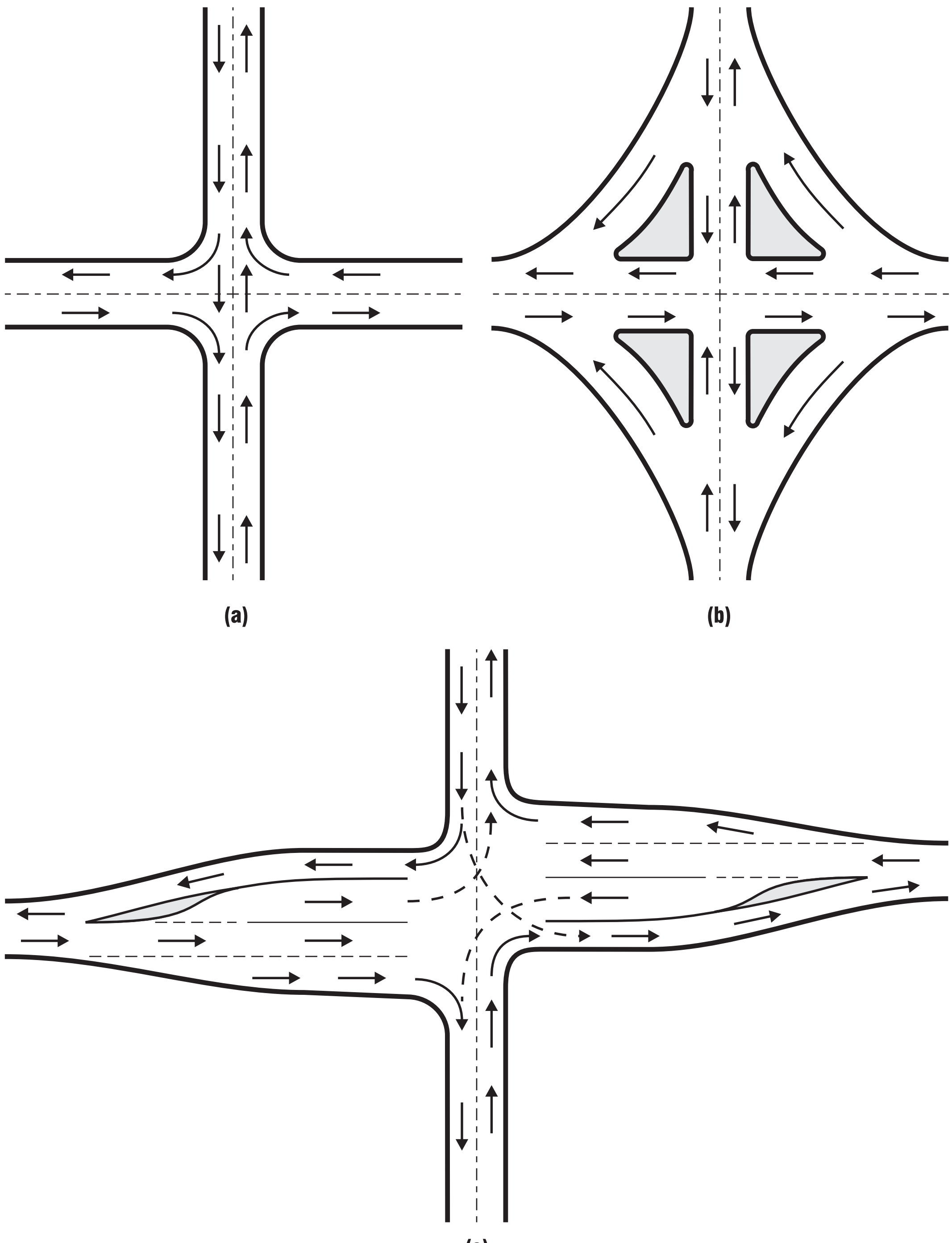

Fig. 16.25 Types of at-grade four-leg intersections: (*a*) unchannelized; (*b*) channelized; (*c*) flared.

the major highway at an intersection should be extended through it and the profile of the minor crossing road adjusted to match that of the major highway at the intersection. This may require transitioning or warping the cross section of the minor road. For low-speed, unchannelized intersections where stop controls or signs are present, it may be desirable to warp the crown of each intersecting roadway. Any alteration of the roadway cross section should be gradual and take into account the effects on drainage.

16.25.4 Islands

An island is a defined area established between traffic lanes in channelized intersections to direct traffic into definite paths. It may consist of curbed medians or areas delineated by paint. In general, islands are provided in channelized intersections to separate and control the angle of conflicts in traffic movement, reduce excessive pavement areas, protect pedestrians and waiting areas for turning and crossing vehicles, and provide a location for traffic-control devices.

16.26 Highway Interchanges

An interchange is a system of interconnecting roadways used in conjunction with one or more grade separations of highways (Fig. 16.26). It accommodates movement of traffic between two or more roadways at different elevations. In so doing, it eliminates grade crossings, which may be unsafe and are inefficient in accommodating both turning and through traffic.When highways carrying high volumes of traffic intersect each other, the greatest degree of safety, efficiency, and capacity is achieved with grade separations of the highways.

There are in use numerous variations of the interchange types shown in Fig. 16.26. They vary in size and magnitude depending on the environment and scope of service for which they are intended. Design of an interchange is based on traffic volume, topography of the site, economic considerations, and environmental factors.

16.26.1 Justification of Interchanges

An interchange is not needed at every highway intersection. The American Association of State Highway and Transportation Officials (AASHTO) considers the following as warranting investigation of the advisability of selecting an interchange instead of an at-grade intersection: highway classifications, need for eliminating traffic bottlenecks and hazards, road-user benefits, and traffic volume.

Design Classification ■ When a highway has been designated to serve as a freeway (Art. 16.1.1), the designer must decide whether each intersecting highway should be terminated, rerouted, or connected to the freeway with a grade separation or an interchange. The goal should be maintenance of a safe and uninterrupted flow of traffic on the freeway. When traffic on an intersecting road must cross the freeway, a grade separation is necessary to eliminate interference with traffic flow on the freeway. When access from the other road to the freeway is required, an interchange is required.

Elimination of Bottlenecks ■ A general drawback to at-grade intersections is the potential, due to high volume of traffic, for spot congestion or bottlenecks occurring at one or more of the approaches. If an at-grade interchange cannot satisfy the capacity requirements of the intersecting highways, then use of some form of interchange should be investigated.

Elimination of Hazards ■ The occurrence of numerous accidents at an at-grade intersection may warrant construction of an interchange. Its feasibility, however, depends on the environment in which the intersection exists. An interchange necessitates acquisition of large amounts of right-of-way. Availability and cost of the needed land is an important consideration in deciding on an interchange. As a result, interchanges are more likely to be employed in rural areas than in urban areas for elimination of hazards.

Road-User Benefits ■ Substitution of an interchange for an at-grade intersection can often lead to direct economic benefits for users. Delays and congestion at an at-grade intersection can be costly because of damage from accidents and the consumption of fuel, tires, oil, and time while waiting for an opportunity to cross or make turns or for signal changes. Time lost waiting at traffic signals can be extremely severe when traffic volumes are

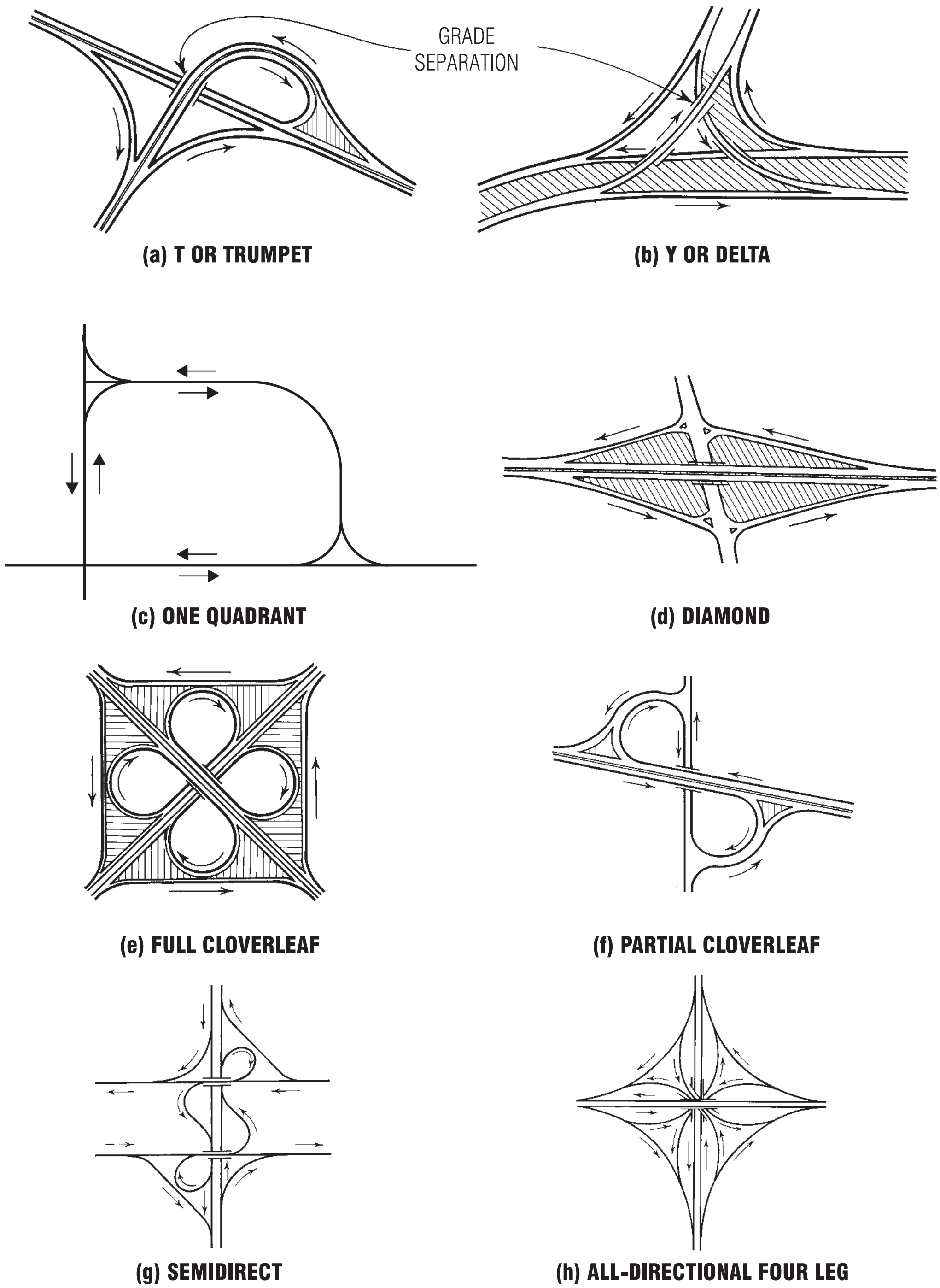

Fig. 16.26 Types of interchanges for intersecting grade-separated highways.

large. Although an interchange requires users to travel a longer distance than they would at a conventional at-grade intersection, this disadvantage is more than offset by the benefits from savings in time resulting from use of the interchange.

To determine whether road-user benefits justify construction of an interchange, the designer should compare the projected benefits with the cost of improvement required. This may be done with the use of the ratio of the annual user benefit to the annual capital cost of the improvement. The annual benefit is the difference between user cost for the existing condition and the cost for the condition after improvement. The annual capital cost is the sum of the annual interest and amortization for the cost of the improvement. The larger the benefit-cost ratio, the greater is the justification for the interchange based on road-user benefits. A general rule of thumb is that a ratio greater than 1 is the minimum required for economic justification. Another consideration is that an interchange can be implemented in stages, in which case incremental benefits can be realized that compare even more favorably with incremental costs.

Traffic Volume ■ While a high volume of traffic is not sole justification for an interchange, it is a major consideration in the overall decision-making process. This is especially the case when traffic volumes exceed the capacity of an at-grade intersection, in which case use of an interchange is generally indicated. The unavailability or high costs of land for an interchange, however, may override the benefits accruing from elimination of the traffic conflicts associated with an at-grade intersection.

16.26.2 Types of Interchanges

After deciding to specify an interchange for a highway intersection, designers have a wide variety of interchange layouts from which to choose (Fig. 16.26). The type of interchange to use and its application at a given site depend on many factors including the number of intersection legs (parts of highways radiating from the intersection), anticipated volume of through and turning movements, topography of the site, culture, design controls, signing, and initiative of the designer.

Design of an interchange typically is custom fit to site conditions and constraints. It is desirable, however, to provide a certain degree of uniformity in interchanges to prevent driver confusion. Also, although interchanges offer greater safety than do at-grade intersections, there are safety issues of concern with interchanges, such as proper signing and placement of exits.

Three-Leg Interchanges ■ These consist of one or more highway grade separations with three intersecting legs. All traffic moves over one-way roadways. In plan view, the roadway layout generally resembles a T or a Y, or delta. A T, or trumpet, interchange is a three-leg interchange in which two of the three legs form a through road and the angle of intersection with the third leg is about 90° (Fig. 16.26*a*). When all three intersection legs are through roads, or the intersection angle of two legs with the third leg is small, the interchange is called a Y, or delta, interchange (Fig. 16.26*b*). Any basic interchange pattern, regardless of through road characteristics or intersection angle, can be adapted to specific site conditions.

Four-Leg Interchanges ■ These consist of one or more highway grade separations with four legs. General categories of four-leg interchanges include ramps in one quadrant, diamond, full cloverleaf, partial cloverleaf, and semidirect- and direct-connection interchanges. Partial cloverleafs include interchanges with ramps in two or three quadrants.

Interchanges with ramps in one quadrant (Fig. 16.26*c*) are generally used where low-volume roads intersect and topography necessitates incorporation of some form of interchange. With such interchanges, turning traffic is facilitated through the use of a single two-way ramp of near-minimum design. Since interchanges are rarely used in areas with a low volume of traffic, application of this type of interchange is somewhat limited. A possible use of a ramp in only one quadrant is for the intersection of a scenic parkway and a state or county two-lane highway. For such a setting, preservation of the existing topography, absence of truck traffic, and relatively small number of turning movements would justify this type of interchange. To control turning movements, however, left-turn lanes must be provided on the through roads and a high degree of channelization is required at the terminals and the median.

Diamond Interchanges. One of the more common types of interchange used, diamond interchanges are generally employed where a highway with large traffic volume crosses but is separated by a bridge from a road carrying comparatively light or slow-speed traffic (Figs. 16.26*d* and 16.27). The diamond layout is the simplest form of all-movements interchange. The two highways are connected by four one-way ramps that may be straight or curved to suit the existing topography or site conditions. The ramps connect with one of the highways at a flat angle.

If the roads carry moderate to large traffic volumes, ramp traffic may be regulated through the use of signal-controlled ramp terminals. When this is the case, widening may be required at the ramp or at the cross street through the interchange area, or at both locations. Each ramp terminal at the minor road is formed with a T or Y at-grade intersection, which allows one left and one right turning movement. If the volume of traffic is large enough, the cross street may be divided and separate lanes provided for the left turns.

A diamond interchange has many advantages over a partial cloverleaf (Fig. 16.26*f*). Unlike a cloverleaf design where traffic typically slows when entering the ramp, diamond interchanges allow entry and exit at relatively high speeds. Also, they occupy a comparatively narrow band of right-of-way, which may not be more than that required for the highway proper.

Split-Diamond Interchanges. These consist of two pairs of parallel or nearly parallel streets connected by two pairs of ramps (Fig. 16.27). As indicated in Fig. 16.27*a*, which shows a split-diamond interchange for two-way streets, the parallel streets need not be consecutive. Figure 16.27*b* is an example of a split-diamond interchange for one-way streets. In the case illustrated, connecting (frontage) roads parallel to the elevated highway are provided between the cross streets to improve traffic movement.

A split-diamond interchange reduces traffic conflicts by accommodating the same amount of traffic at four rather than two crossroad intersections. This has the effect of reducing the left-turn movements at each intersection from two to one. One drawback to the split-diamond interchange, however, is that traffic leaving the elevated highway cannot return to the same interchange and continue in the same direction.

Cloverleaf Interchanges. A cloverleaf interchange provides direct connections for right turns between two highways but utilizes loop ramps to accommodate left turns. A full cloverleaf (Fig. 16.26*e*) has loops in four quadrants, whereas a partial cloverleaf (Fig. 16.26*f*) has loops in only two quadrants.

While a cloverleaf interchange greatly reduces accidents by eliminating all left turns, it does possess drawbacks. For example, high speed and large volume of traffic require large radii for the loop ramps and hence acquisition of very large areas of right-of-way. This has greatly limited use of cloverleafs in urban regions. Even a slight increase in design speed can require significantly greater radii. For a design speed of 25 mi/h, for instance, design standards call for a loop radius of 150 ft. An increase of only 5 to 30 mi/h, an increase of 20%, requires a radius of 230 ft, an increase of 53%. Furthermore, the area required for right-of-way increases by about 130%.

Another disadvantage of cloverleafs is that left-turning traffic must travel a greater distance than otherwise would be required and significant weaving movement may be generated. For a loop designed for 20 mi/h and having a 90-ft radius, for example, the extra travel distance required is about 600 ft. In contrast, for a loop designed for 25 mi/h and having a 150-ft radius, the extra distance is roughly 1000 ft, and for 30 mi/h on a loop with a 230-ft radius, the extra distance is about 1500 ft. Moreover, since travel time on ramps varies almost directly with the length of ramp, the time that might be saved by increased speed is lost over the greater distance that must be traversed. In addition, weaving maneuvers associated with the use of a cloverleaf for left turns can result in serious vehicle interference and a corresponding slowdown of through traffic, especially when the flow exceeds 1000 vehicles per hour.

Since it is seldom practical to provide for more than a single lane on a loop, a ramp can be expected to accommodate no more than 800 vehicles per hour. If truck traffic is not anticipated and the design speed for the ramp is 30 mi/h or higher, a design capacity of 1200 vehicles per hour can be used. Thus loop-ramp traffic capacity is a major constraint and can limit the effectiveness of a cloverleaf interchange.

Partial Cloverleaf Ramp Arrangements. A partial cloverleaf interchange utilizes loop ramps in only two or three quadrants. This type of interchange is desirable when the anticipated traffic distribution does not require a full cloverleaf. A major design decision is selection of the quadrants in which ramps should be placed.

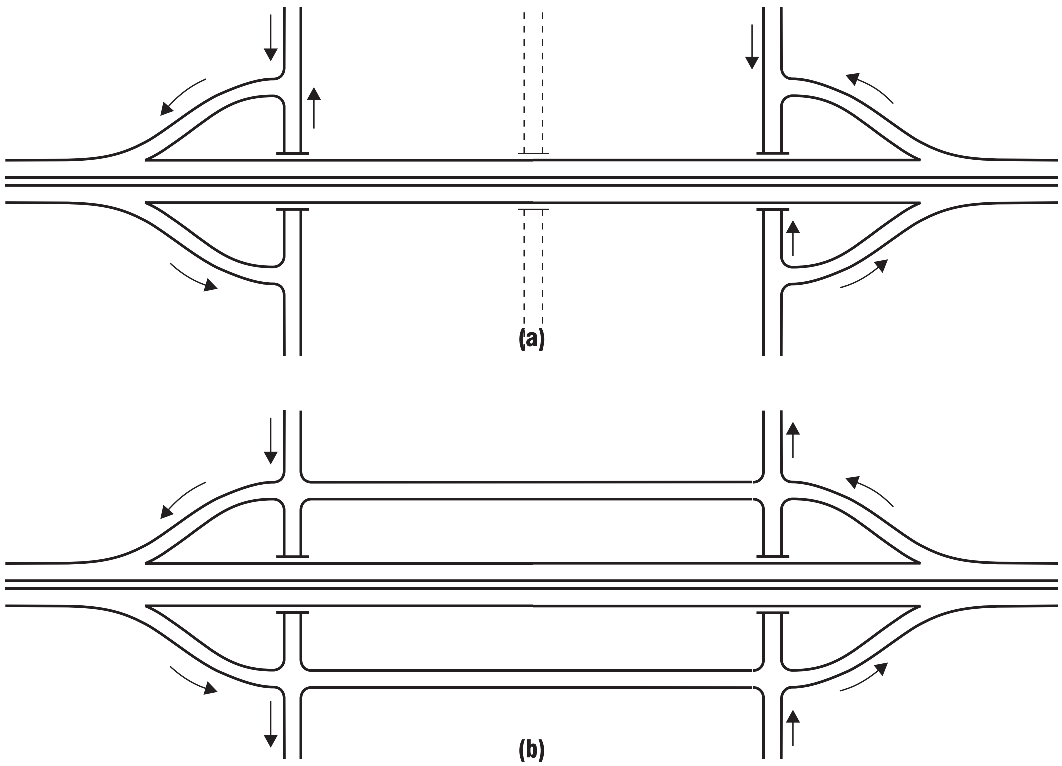

Fig. 16.27 Split-diamond interchanges: (*a*) with two-way streets; (*b*) with one-way streets.

The arrangement of ramps in a partial cloverleaf interchange should facilitate major turning movements at right-turn exits and entrances and limit impediments to traffic flow on the major highway. If traffic on the major highway is much greater than that on the minor intersecting road, right-turn exits and entrances, in general, should be placed on the major highway. Such an arrangement, however, will require a direct left turn off the crossing road.

Directional Interchanges. These provide direct or semidirect connections between intersecting highways. They are often preferred to cloverleaf interchanges, which are composed only of loops and consequently may fail to meet the high speed and traffic volume demands of an expressway.

A **direct connection** is a one-way roadway that does not deviate greatly from the intended direction of travel. An interchange that utilizes direct connections for all major left turns is called a directional interchange (Fig. 16.26*h*). It may also incorporate loops for minor left turns. Loops in conjunction with direct connections are generally used in rural areas where direct connections in all quadrants cannot be justified.

A **semidirect connection** is a one-way roadway that deviates from the intended direction of travel but is more direct than a conventional loop (Fig. 16.26*g*) Therefore, a semidirectional interchange is similar to a directional interchange except that it utilizes semidirect connections to accommodate major left turns.

Directional interchanges typically require several grade separations. Figure 16.26*h* shows a directional interchange with a four-level structure.

Directional interchanges are generally designed to accommodate many site-specific conditions, including topography, geometry, and traffic demands. The design of individual ramps should satisfy accepted standards for curvature, pavement widths, length of weaving sections, and entrance and exit design criteria.

16.26.3 Selection of Interchanges

The type of interchange to select is one that best meets the needs of the site and provides operational efficiency and safety, and adequate capacity for anticipated traffic volumes and turning patterns. It is advisable to choose the type of interchange before final route selection. This permits a determination that the interchange type selected can be adequately developed.

Interchanges generally fall into two basic categories: system interchanges and service interchanges. The former includes interchanges that connect one freeway to another. In contrast, service interchanges connect a freeway to a road with a lower-grade classification. In a rural environment, service demand is a principal issue.

When the intersecting roadways are freeways, all-directional interchanges may be advantageous to facilitate high turning volumes. When only some turning volumes are high, a combination of directional, semidirectional, and loop ramps may prove advantageous. Where directional or semidirectional interchanges are used in conjunction with loops, however, weaving sections should be avoided.

A cloverleaf interchange is the minimum type to use at the intersection of two highways both of which have partial or full control of access. Cloverleafs are also advantageous for intersections where left turns at grade are prohibited. A diamond intersection is appropriate where a major highway intersects a minor road. A partial cloverleaf may be advisable where traffic volumes or site conditions do not warrant or allow employment of a full cloverleaf interchange.

Interchanges for highways in urban areas should be selected on a systemwide rather than individual basis. Since interchanges are usually closely spaced in urban environments, an interchange may influence selection and design of preceding and subsequent interchanges. For example, additional lanes may be required to accommodate capacity, weaving, and lane-balance requirements.

A general rule of thumb is that a minimum interchange spacing of 1 mi should be used in urban areas, but a closer spacing may be used if grade-separated ramps are provided or collector-distributor roads are added. A minimum spacing of 2 mi should be used in rural regions.

16.26.4 Ramps in Interchanges

A ramp is a roadway that connects two or more legs of an interchange and is used for turning traffic (Fig. 16.28). The main elements of a ramp are a connecting roadway and a terminal at each end. The profile of the connecting roadway typically is sloped and the horizontal alignment is curved. In general, design criteria for horizontal and vertical alignments of ramps are less restrictive than those of the intersecting highways, but sometimes the design criteria are the same.

In design of a ramp, the designer has to balance several factors. For example, consider topography and costs of right-of-way, which influence selection and design of the ramp. To conserve land, it may be necessary to locate the ramp so close to the highway that a retaining wall must be constructed. The cost of the wall then has to be balanced against the cost of acquiring additional right-of-way to eliminate need for the wall.

The type of ramp to use depends on the type of interchange. A trumpet interchange, for example, utilizes one loop, one semidirectional ramp, and two right directional or diagonal ramps (Figs. 16.26*a* and 16.28). Usually, a ramp is a one-way roadway. Some ramps, such as a diagonal ramp, are one-way but allow both left and right turns at a terminal on a minor intersecting road.

Ramp Design Speeds ■ The design speed for a ramp generally should be about the same as that for the intersecting highway with the lower traffic volume. Although lower ramp speeds may be necessary, the design ramp speed used should not be less than the values presented in Table 16.9. The table lists, as a guide, ramp design speeds to be used with various highway design speeds.

When a ramp connects a high-speed highway with a minor road or a city street, provision should be made for a considerable reduction in speed for traffic leaving the high-speed highway. An initial speed reduction can be accomplished through use of a deceleration lane on the main highway. To allow for continuing deceleration on the ramp, the radii of the curves on the ramp should be reduced in stages. At the ramp terminal at the minor road, it may be necessary to provide some form of signal or signing to stop or slow vehicles.

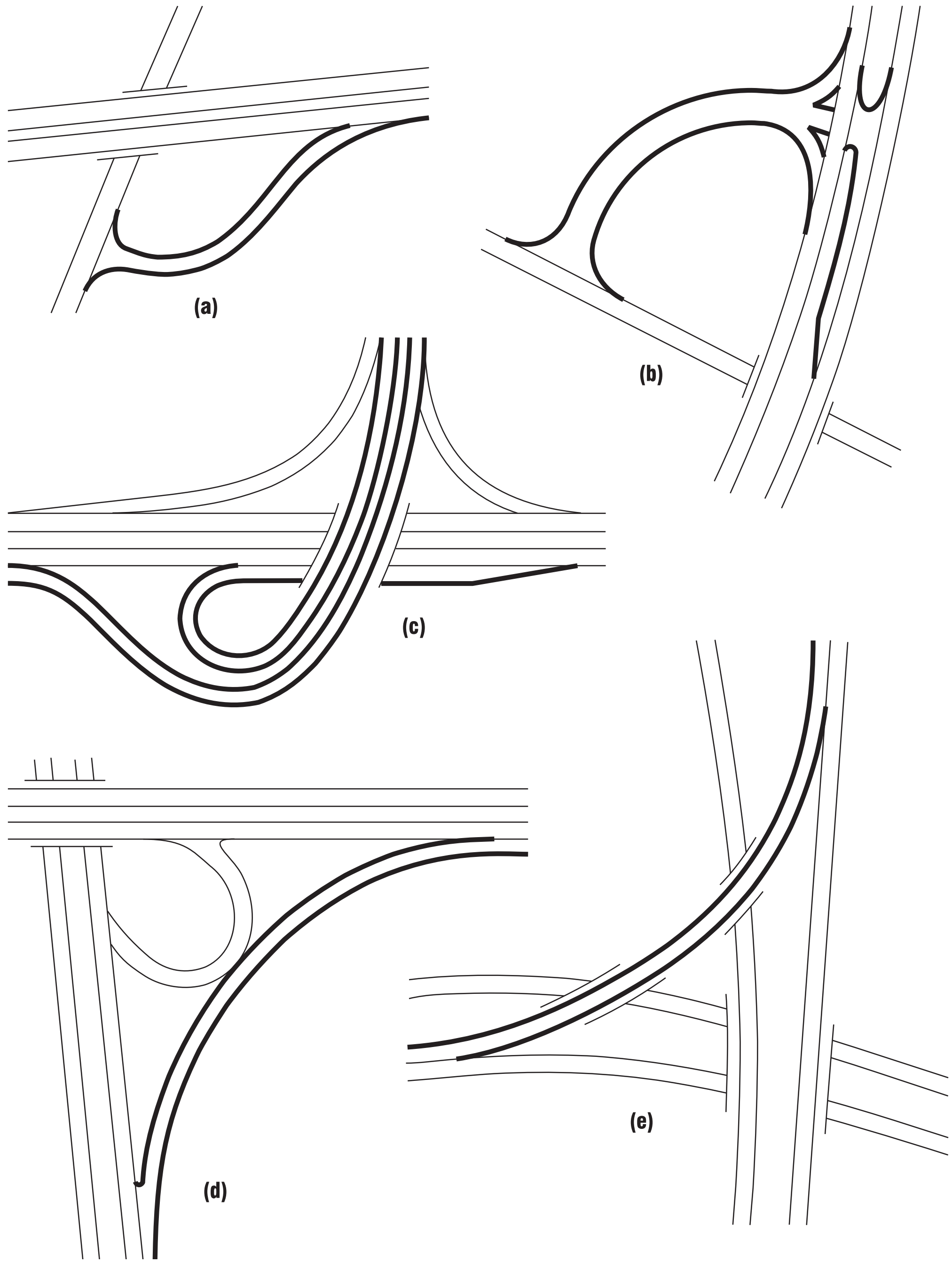

Fig. 16.28 Types of ramps: (*a*) diagonal; (*b*) one-quadrant; (*c*) loop and semidirect; (*d*) outer connection; (*e*) directional.

Table 16.9 Suggested Ramp Design Maximum Speeds, mi/h, Based on Design Speeds of Connected Highways*

Ramp design speed, mi/h	Highway design speed, mi/h					
	30	40	50	60	65	70
Upper range (85%)	25	35	45	50	55	60
Middle range (70%)	20	30	35	45	45	50
Lower range (85%)	15	20	25	30	30	35

* Adapted from "A Policy on Geometric Design of Highways and Streets," American Association of State Highway and Transportation Officials.

Ramp Curvature ■ The principles governing horizontal curvature (Art. 16.13) are also applicable to the design of interchange ramps. For example, use of compound curves and spirals is often beneficial in adapting a ramp to site-specific conditions and providing a natural path for vehicles. Loops, except for their terminals, may be composed of circular arcs or some other curve that is formed with spiral transitions.

Ramp Sight Distance ■ Safety demands provision for adequate sight distance along ramps and at the ramp terminals. Sight distance along ramps should be at least as long as the safe stopping sight distance. Sight distance for passing, however, is not required.

At the ramp terminals, a clear view of the entire exit terminal should be provided. The exit nose and a section of the ramp pavement beyond the gore, the area downstream from the shoulder intersection points, should be clearly visible

Ramp Vertical Curves ■ In general, a ramp grade should be as flat as practical to limit the amount of driving effort needed to traverse from one road to another. A ramp profile typically resembles the letter S. It consists of a sag curve at the lower end and a crest curve at the upper end. When a ramp goes over or under another roadway, however, additional vertical curves may be required.

Ramp Terminal ■ This is the portion of a ramp that adjoins the through traveled way. The terminal includes speed-change lanes, tapers, and islands. A ramp terminal may be an at-grade type, as is the case for a diamond interchange, or a free-flow type that allows ramp traffic to merge with or diverge from high-speed through traffic. For the free-flow type, the intersection with the through traffic should be made at a relatively flat angle. The number of lanes on the ramp at the terminal and their configuration also influence the type of ramp terminal to be used and its associated design.

Traffic Control and Safety Provisions

To design roads that are safe and efficient in accommodating traffic flow, highway engineers should be familiar with the basic characteristics not only of roads but also of drivers and vehicles. In addition, these engineers should have knowledge of highway-related causes of accidents and means of avoiding them. To reduce the number of highway accidents, a multiple approach is necessary, including improvements in driver and pedestrian training and education, law enforcement, vehicle design, and highways. A very high percentage of highway accidents result from driver error, often associated with law violations. Good highway design, nevertheless, can help prevent accidents. Statistics indicate that the relative frequency of accidents associated with vehicle movements or maneuvers depends to a great extent on the type of highway and, in particular, on various design features that are intended to prevent traffic conflicts. Many features that are advantageous in accident avoidance are discussed in preceding articles. Other features, such as traffic control devices and highway lighting, are discussed in the following.

16.27 Traffic Control Devices

These provide for the safe and orderly movement of traffic on a highway by offering guidance and navigation information to drivers. Commonly used traffic control devices include traffic signals and signs that display regulatory warnings and route information. Other forms include pavement markings and delineators.

An effective traffic control device should command attention, convey a clear and simple meaning, acquire the respect of drivers, and allow adequate time for drivers to respond. Traffic control devices should be uniform, treating similar situations in the same fashion. Consistent use of symbols and location of signs and other traffic control devices helps to give drivers sufficient response time for reacting to traffic messages.

16.27.1 Traffic Signs

In general, traffic signs may be classified as regulatory, warning, or guide. Regulatory signs are used to indicate the required method of traffic movement. Examples of regulatory signs include STOP and YIELD signs used on intersecting roadways to establish the right of through movement. Warning signs, such as a FALLEN ROCK ZONE sign, are used to indicate potentially hazardous conditions. Variable message signs are used under remote control or automatic sensors, among other purposes, to convey emergency warnings. Guide signs, such as exit signs on a freeway, are used to direct traffic along a route toward a destination.

Placement and design of signs should be an integral part of highway design, especially in preparation of highway geometry. Such an approach will help ensure that future adverse operational conditions will be minimized or eliminated.

Signs are typically manufactured from light-reflective materials. In areas of high traffic and in construction zones, illuminated signs are often used.

16.27.2 Delineators

These are reflectors that are used to guide traffic, especially at night. They may be mounted above-ground or fixed to the pavement. In the latter case, delineators may act as a compliment to or replacement of conventional pavement markings (Art. 16.27.3) and are known as raised pavement delineators. Because they are subject to uprooting by snowplow blades, use of this type of delineator as a permanent marker is more predominant in warm climates than in cold ones. Raised pavement delineators, however, are commonly used in any environment as construction zone markers to delineate temporary travel lanes.

When mounted on a post, delineators are reflectors typically made of faceted plastic or glass. These units are installed at specific heights and spacings to delineate the horizontal alignment, typically in regions where alignment changes may be confusing or ill-defined. Delineators at interchanges are usually different in color and multiple-mounted to differentiate the interchange area from the roadway proper.

16.27.3 Pavement Markings

Pavement markings are markers that are on the roadway surface and that are used to regulate and guide movement of traffic in a safe, orderly, and efficient manner. The forms of pavement markings include centerline stripes, lane lines, no-passing barriers, and edge striping. Painting is the most common method of applying pavement markings. An alternative is plastic striping fixed to pavement with an adhesive. This method is often used for marking temporary lanes. For the pavement markings to fulfill their intended functions, they must be visible, and for this, they must be properly maintained by cleaning and renewal when required.

16.27.4 Traffic Signals

Traffic signals are used to assign the right-of-way at intersections and thereby control the flow of vehicular and pedestrian traffic. Signals can also be used to emphasize a hazardous location, supplement conventional signs, and provide control at railroad-highway grade crossings.

Red, yellow (amber), and green signal lights are widely used. Depending on the type of intersection, the displays may have a circular or arrow configuration. Care should be taken in placement of traffic signals to ensure visibility, meet pedestrian requirements, and effect integration of the signals with the highway geometry. The design of a traffic signal system should also allow for future expansion.

Traffic signals may be pretimed, traffic-actuated, or pedestrian-activated. Pretimed signals operate on a predetermined, consistent, and regularly repeated sequence of intervals. Traffic-actuated controls utilize some form of vehicle and pedestrian detector that trips the signal to allow assigned movements. Typical uses of traffic-actuated signals are to control left turns and traffic flow from side roads, movements that are not permitted until a vehicle trips, or actuates, a time-delay detector. Pedestrian-actuated signals allow normal vehicular movement until a pedestrian presses a button that changes the signal light, halting traffic and allowing the pedestrian to cross safely.

Signal Systems ■ These are used to coordinate the movement of traffic through intersections on major highways located in and on approaches to cities and large villages. In signal systems, a master controller resynchronizes various intersectional signal controllers to reduce the inconvenience and delays resulting from independent control of traffic signals in cities and large villages.

16.27.5 Colored Pavement

Another method of delineating pavement sections to guide and regulate traffic is to color sections of the pavement, such as shoulders (Art. 16.5). In order for colored pavement to serve successfully as a traffic control device, it should provide significant contrast with adjoining paved areas.

16.27.6 Ramp control

It is sometimes necessary to control entry of vehicles to limited-access highways from ramps. This purpose may be achieved in several ways. One way is to close the ramps. This involves complete diversion of ramp traffic. Another way is to apply ramp metering. This requires drivers to stop and wait before entering the highway when ramp traffic flow must be restricted. An alternative is merge control, which employs a ramp-metering system that stops vehicles at the ramp terminal when there is steady traffic on the highway and releases them when the system detects a gap in the highway traffic.

16.27.7 Traffic Surveillance and Control Systems

These utilize video and related equipment to monitor traffic manually. The objective is to keep traffic flowing as orderly and efficiently as possible. Traffic surveillance and control systems can range from limited types that use conventional detectors to elaborate systems that employ vehicle detection loops, helicopters, and video equipment.

The goal of traffic surveillance and control systems is to provide from remote locations observation of traffic movements and permit immediate identification of demands for service. These systems can also play an invaluable role in promoting highway safety by facilitating immediate recognition of emergencies. In such situations, the controllers can promptly notify the proper authorities who can take appropriate actions to dispatch aid to the scene. The controllers also can initiate management of traffic flow through the use of variable message signs (Art. 16.27.1).

16.28 Intelligent Vehicle Highway Systems

A major intent of the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991 is to promote research and development in interfacing of highways with other forms of transportation, including railroads, aviation, shipping, and mass transit. A key component of ISTEA is its emphasis on innovation and new technologies, such as intelligent vehicle highway systems (IVHS) and magnetic levitation systems (MAGLEV) for railways.

An IVHS is a collection of systems that address a variety of different objectives. Some elements of an IVHS, like variable message signs, are commonplace. Still others, such as in-vehicle displays and traveler information screens, are gradually being adopted.

16.18.1 Advanced Traffic Management Systems (ATMS)

This is a component of an IVHS that has the ability to detect accidents, construction work, and other causes of traffic backup and congestion. The ATMS also may offer alternate paths for vehicles in an effort to dispel congestion and provide optimal use of the open highway system.

An ATMS is essentially a form of traffic surveillance and control system (Art. 16.27.7). Two important aspects of an ATMS are the type and location of detection equipment used to identify points of incident. An ATMS requires robust detectors in order to provide necessary information. Loop detectors are often used in IVHS, since they generally are less expensive and more reliable than currently available video-image processing. Loop detectors alone, however, do not provide all the information needed for proper management of traffic congestion. They require supplementing by other technologies, such as closed-circuit television (CCTV), which can provide personnel located in a control center with more specific information concerning incidents. Typically, implementation of CCTV is confined to major highways and bottleneck points on minor roads.

16.28.2 Advanced Traveler Information Systems (ATIS)

These are components of an IVHS that provide drivers with in-vehicle navigational information and real-time data concerning the location of existing or potential traffic conflicts. An ATIS may also suggest alternative travel routes.

16.28.3 Advanced Vehicle Control Systems (AVCS)

These are components of an IVHS that are designed to give advance warning to drivers of potential collision with other vehicles. An AVCS may be able automatically to brake vehicles if a collision is imminent or direct vehicles away from a potential collision.

16.28.4 Commercial Vehicle Operation (CVO)

These are components of an IVHS that monitor movements of trucks, buses, vans, taxis, and emergency vehicles. Tracking of commercial vehicles has many benefits. One benefit is that CVO facilitates automation of toll collection, which greatly helps in reducing congestion at toll collection facilities. Another benefit is the ability to track movements of hazardous material and of large vehicles that exceed legal truck-load limitations.

16.28.5 Advanced Public Transportation Systems (APTS)

In addition to benefiting users of highway, an IVHS is also designed to benefit the users of mass transit or high-occupancy vehicles (HOV) through incorporation of an advanced public transportation system. An HOV may be a bus, van with more than one passenger, or car pool. The goals of an APTS are to reduce operating costs for transit systems and promote use of transit systems through increased efficiency. An APTS allows transit riders to prepay fares and receive in evidence of the payment a *smart* card that can be used to gain access to transit vehicles.

16.29 Highway Lighting

Nighttime illumination of a roadway is very important in promoting safety and operational efficiency. As with any highway appurtenance, however, there is an associated cost that should be balanced against the enhancement offered.

In general, lighting is used more extensively for urban rather than rural roadways. In addition to furthering highway safety, lighting in urban environments promotes safety to pedestrians. In rural areas, lighting is generally applied in critical areas, such as interchanges, intersections, railroad crossings at grade, narrow or long bridges, tunnels, sharp curves, and areas where roadside interference is a concern.

A typical highway lighting installation consists of an aluminum or steel standard (pole) on top of which is mounted a luminaire (Fig. 16.29). This lighting fixture comprises a lamp, its housing, and a lens.

Like other roadside elements, lighting standards are susceptible to vehicle impact and therefore should be placed outside the roadway clear zone. If it is not possible or practical to locate the standards in a *safe* area, the standards should be equipped with some form of impact-attenuation feature. For this purpose, breakaway poles may be used. They should be installed along stretches of roadway where vehicles should be traveling at relatively low speeds at which damage to a vehicle striking a standard will not be severe. Breakaway poles should not be used, however, in heavily developed regions, where there is the possibility of an impacted pole damaging adjacent buildings or pedestrians.

Installation of poles on the outside of curves on a ramp should also be avoided because in such locations they are likely to be struck by a vehicle. If lighting standards are placed behind a longitudinal traffic barrier, they should be offset to allow for deflection of the longitudinal barrier when it is impacted.

When installed for a divided highway, lighting standards may be placed either in the median (Fig.

16.29*a*) or on the right side of the road (Fig. 16.29*b*). For high-speed lanes, it is generally preferable to place the standards, equipped with dual-mast arms, in the median since the cost is typically lower and the illumination provided greater than for standards on the right side of the roadways. Overhead lighting installations, such as those depicted in Fig. 16.29, generally extend 30 ft above grade and are equipped with mercury-vapor lamps.

For interchanges, traffic circles, and toll plaza areas, another form of overhead lighting, known as high-mast lighting, is used. In this case, luminaires are mounted on tapered steel poles or triangular steel towers that range in height from 50 to 150 ft. The luminaires can be lowered to within 3 ft of the ground for periodic inspection and maintenance. To further facilitate maintenance, hoisting and electric cables can be replaced at ground level, where electrical connections are made. The lamps typically are 1000-W mercury vapor, metal halide, or high-pressure sodium vapor.

Even if initially the design of a highway does not specify highway lighting, provision for future installation of lighting should at least be considered. If lighting should be required in the future, its installation will be greatly facilitated by provi-

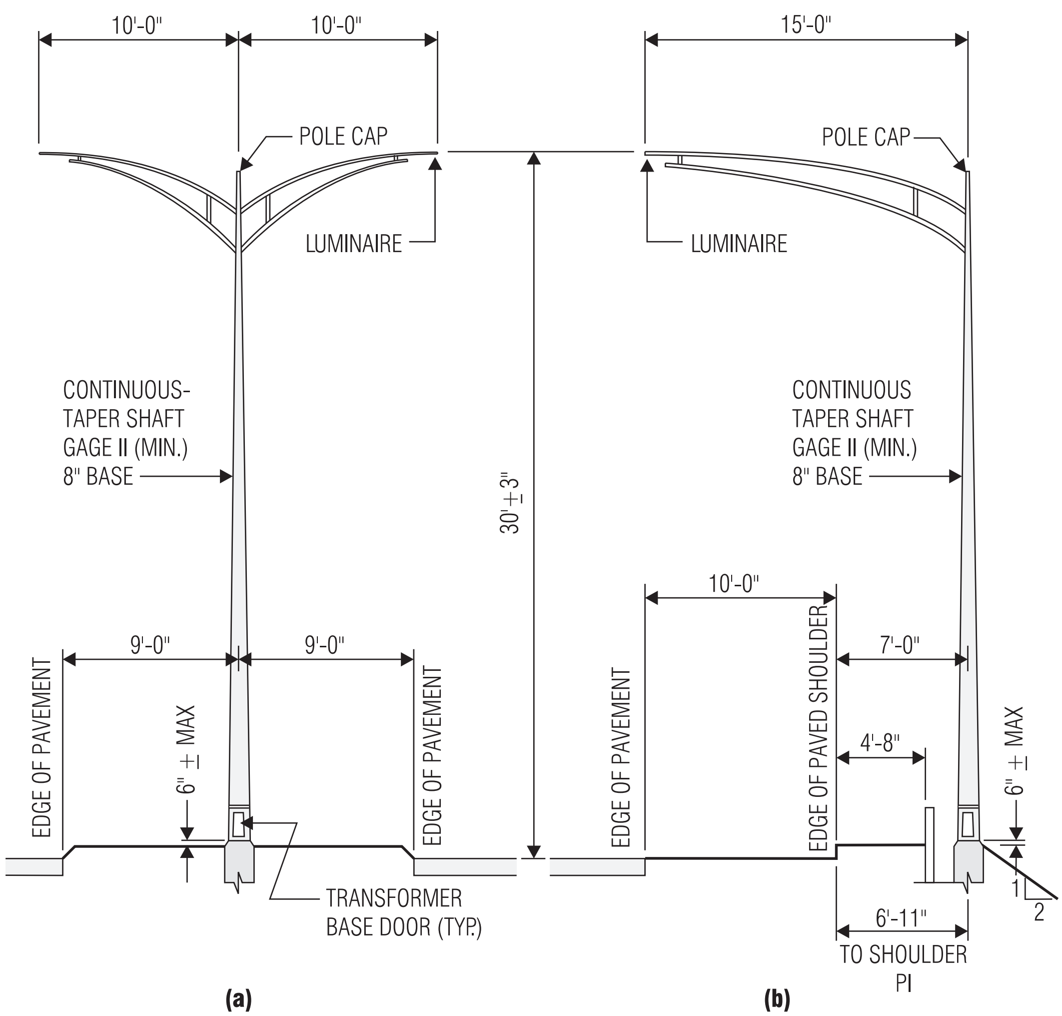

Fig. 16.29 Highway lighting installation with luminaires mounted on tall posts.

sion of the necessary conduits under pavements and curbs during construction of the highway.

("An Informational Guide for Roadway Lighting," American Association of State Highway and Transportation Officials, Washington, D.C.)

Highway Maintenance and Rehabilitation

Maintenance and rehabilitation of highway pavements are ongoing activities, critical for prolonging the life of highways. Methods for performing these tasks vary from region to region and depend on the type of pavement.

16.30 Maintenance of Asphalt Pavements

Deterioration of an asphalt pavement is evidenced by distortion and various forms of cracking.

16.30.1 Distortions of Asphalt Pavements

A distortion is a change of a pavement from its original shape. Asphalt pavements can suffer from a variety of distortions that can cause cracking and other adverse conditions. The principal forms of pavement distortions are channeling (rutting), corrugations, shoving, depressions, and upheaval.

Channeling is a lengthy depression formed in wheel tracks. **Corrugation** (washboarding) is the plastic movement of an asphalt surface that causes formation of ripples across the pavement. **Shoving** is plastic movement that results in a localized bulge in the pavement. **Upheaval** is the localized upward displacement of a pavement brought on by swelling of the subgrade or other portion of the pavement structure.

16.30.2 Cracking of Asphalt Pavements

This has many causes and takes a variety of forms, such as alligator, edge, joint, reflection, slippage, transverse, longitudinal, and diagonal cracking.

Alligator Cracking ■ Cracks that form small interlocking rectangular shapes having the appearance of alligator skin are known as alligator cracks. These usually are initiated by failure of an untreated granular base or by a soft subgrade layer. Such conditions often arise from excessive saturation of pavement base or subgrade.

Maintenance generally involves removal of all wet material and patching with a full-depth hot-mix asphalt. To prevent future occurrence of alligator cracking, new drainage facilities should be installed or existing drainage facilities should be improved (Arts. 16.16 and 16.17).

Edge Cracking ■ Located at or near the edges of pavements, edge cracking extends longitudinally, nearly parallel to the centerline of the roadway. This type of cracking may be accompanied by transverse cracks, nearly perpendicular to the roadway centerline. Causes of edge cracking include settlement of the pavement, inadequate support for the pavement, inadequate drainage, and frost heave.

Repair of edge cracks requires filling the cracks with asphalt-emulsion slurry or cutback asphalt mixed with sand. If settlement has occurred, it may be necessary to bring the roadway surface to grade through use of hot-mix asphalt patching.

Joint Cracking ■ This occurs at the interface between a pavement and adjacent shoulder. Joint cracking can be initiated by deformational loads due to thermal expansion and contraction or alternate wetting and drying. It also can be caused by intrusion of water as a result of inadequate drainage.

A crack between two adjacent paving lanes is known as a **lane-joint crack**. It typically is caused by inadequate bond or a poor seam between adjoining sections of pavement.

Repair of joint cracks requires filling of the cracks with an asphalt-emulsion slurry. In addition, poor drainage conditions should be corrected.

Reflection Cracking ■ This is a crack that forms in an asphalt overlay and reflects the pattern of the underlying pavement surface. Reflection cracking can be induced by horizontal or vertical movements in the pavements beneath the overlay. These movements are generally caused by traffic loads, earth movement, or temperature. Reflection cracks typically occur in asphalt overlays placed on top of a portland cement concrete or cement-treated base.

Cracks less than 1/8 in wide may either be ignored or, if intrusion of water is a concern, filled, with the use of a squeegee, with emulsified or cutback asphalt covered with sand. Cracks more than 1/4 in wide should be filled with an asphalt-emulsion slurry or a light grade of cutback asphalt and fine sand.

Shrinkage Cracking ■ This is evidenced by interconnected cracks that create a series of large blocks with sharp corners or angles. Shrinkage cracks are usually associated with a volume change in the pavement asphalt mix, base, or subgrade. They also may result from aging of the pavement. The constant exposure of the pavement materials to thermal expansion and contraction, may cause them to lose some of their elasticity or resiliency and bring about shrinkage cracking.

Slippage Cracking ■ These are crescent-shaped cracks generated by traffic-induced horizontal forces. Slippage cracks are caused by insufficient bond between the surface layer and the underlying course. Dust, dirt, and oil atop the underlying course during placement of the surface course can contribute to this lack of bond. Also, omission of a tack coat during construction can lead to formation of slippage cracks.

This type of cracking is repaired by removing the surface layer around the crack to locations where an adequate bond is present. The area from which the surface course was removed is then patched with a hot-mix asphalt.

("The Asphalt Handbook," The Asphalt Institute, College Park, Md. See also Art. 16.32.)

16.31 Maintenance of Portland Cement–Concrete Pavements

Deterioration of a portland cement–concrete pavement (PCC) is evidenced by distortion and various forms of cracking. Consequently, much maintenance work is concerned with filling of cracks and expansion joints. For this purpose, asphalt is often used. It is suitable for sealing joints and cracks, filling small cavities, and raising sunken slabs. A more extreme alternative in maintaining PCC pavements is to cover deteriorated pavement with a thin asphalt course (overlay).

16.31.1 Distortions of PCC Pavements

Major forms of distortion in PCC pavements are faults and pumping. A fault is a physical difference in elevation between two slabs located at a joint or a crack. Pumping is the upward and downward movement of a slab under traffic loads. This may occur when pavements overlay very wet sand, clay, or silt. Pumping typically takes place at transverse and longitudinal joints and edge cracks. It may be corrected by inserting asphalt or portland cement grout under the slab and improving the drainage.

16.31.2 Cracking of PCC Pavements

This has many causes and takes a variety of form, such as transverse, longitudinal, and diagonal cracking.

Transverse Cracking ■ Extending roughly perpendicular to the pavement centerline, transverse cracking may be caused by overloading of the pavement, pumping of slabs, failure of a soft foundation, frozen joints, lack of joints, excessively shallow joints, or concrete shrinkage. Repair usually requires that the cracks be cleaned of all loose material by routing, compressed air, or sandblasting and then filled with a rubber-asphalt sealer. Cracks generated by pumping should be provided with an asphalt underseal.

Longitudinal Cracking ■ This extends roughly parallel to the pavement centerline. Longitudinal cracking can be caused by shrinkage of the concrete, especially in wide pavements without a longitudinal joint. Other conditions that can create longitudinal cracking are pumping or an expansive subbase or subgrade.

Repair of longitudinal cracks in PCC pavements is the same as that for transverse cracking. For pumping-induced cracks, a high-softening-point asphalt may be used to fill the voids under the pavement slab.

Diagonal Cracking ■ These run diagonally to the pavement centerline. They are induced by traffic loads on an unsupported end of the pavement slab.

Repair of the cracks is like that described for transverse and longitudinal cracking. As may be done for pumping-type cracks, an asphalt underseal may be applied to the slab and followed by filling of the crack with a rubber-asphalt sealing compound.

See also Art. 16.32.

16.32 Pavement Management Systems (PMS)

Constant exposure to the elements, combined with wear and tear from traffic, make highways extremely prone to deterioration. As a result, they must be repaired or replaced if they are to serve as intended. Highway maintenance and rehabilitation, however, is not limited just to application of the remedial measures described in Arts. 16.30 and 16.31.

Responsible for maintenance of immense lengths of roadway and associated appurtenance facilities, transportation agencies frequently have to decide which sections of highway need immediate attention and which can be deferred. A variety of factors influence this decision, and highway design is only one of these factors, albeit an important one. Economic, political, and a host of other factors must be evaluated before project selection may be made. The main objective of a pavement management system (PMS) is to assist in making this decision. The human component of a PMS is essential in the decision-making process, but proper use of computer software can play an important role in the decision-making process.

The basic components and associated products of a PMS are as follows: inventory database, maintenance database, budgetary information, project selection methods, and costing models.

The inventory database of a PMS details pavement conditions throughout the entire highway network. There are many ways to define the state of a section of pavement. One method is to rate the pavement in terms of various forms of pavement distress, such as edge cracking and rutting, as described in Arts. 16.30 and 16.31. The length of the pavement section to be rated depends on the detail desired. Use of small lengths, however, does not necessarily translate into a more accurate picture of pavement condition.

Typically, information on pavement condition is stored in a computerized database management system (DBMS) for both querying and modeling. The data may also be tied into a geographic information system (GIS), which allows excellent visualization of data. Historical data contained within the maintenance database describe what work has been performed on the pavement sections. The data are helpful in determining both the results of individual remedial methods and associated costs. Budgetary information may be derived from the inventory and maintenance databases. Based on the data thus made conveniently available, project selection and cost analysis methods can be applied to assist in selection of the sequence in which projects are to be implemented and formulation of highway maintenance and rehabilitation budgets.

16.32.1 Project and Network Level Analyses

A PMS can function using a project or network level analysis approach, or both. Project level analysis deals with individual sections of pavement and the remedial measures to be taken to correct deficiencies. Associated cost estimating can be performed and ramifications of various remedial measures can be predicted with the objective of determining which method and level of repair will yield the best results in terms of both economy and safety.

Network level analysis is applicable to a group of projects comprising various sections of noncontiguous highway. This analysis permits formulation of alternatives based on the maintenance and rehabilitation needs not only of specific highway sections but also of the highway network as a whole. For example, one section of the network may require patching of alligator cracking and another may show evidence of subsurface drainage inadequacies. If funds are insufficient for correcting both conditions, the PMS could assist in the decision whether to correct the drainage condition, which if ignored could lead to failure of the pavement, or to patch the cracking, which should not be ignored but may be deferred for a short time without serious consequences. While this is a relatively simple example, it serves to illustrate the basic concepts behind network level analysis.

In addition to providing analysis, the PMS offers valuable support information in the form of cost and record data and ancillary backup information that can be used not only for formulating but also for justifying maintenance plans. Development of a PMS builds upon methods and information currently in use in an effort to create an integrated system for planning and performing pavement maintenance and rehabilitation.

16.32.2 Predicting Future Pavement Condition

In addition to assisting in selection of projects for repair, a PMS may be used to predict the future condition of pavement. Predictions are typically based on one of the following assumptions: no repair work is performed; partial, interim remedial measures are taken; or full repairs are made to correct all deficiencies.

The estimates of future pavement conditions provide maintenance planners with a more accurate picture of the ramifications of various options than could otherwise be obtained. This information is also useful in developing long-range plans and estimating future costs.