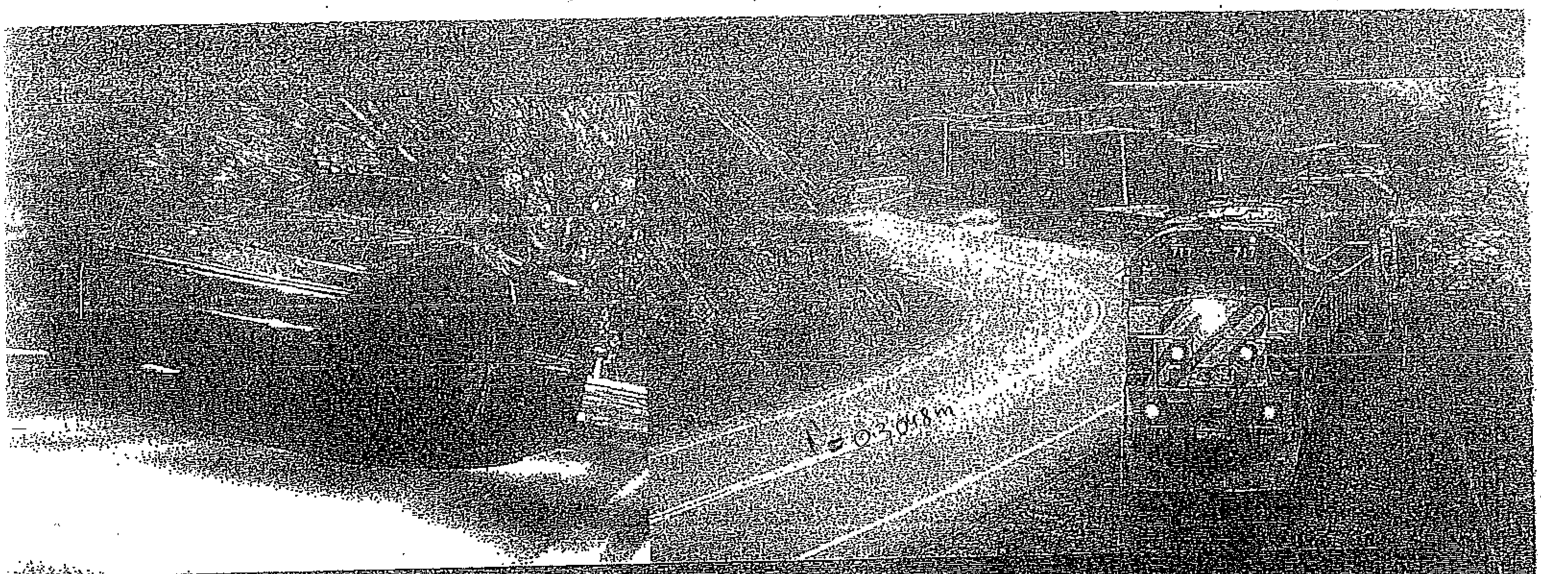

TRANSPORTATION ENGINEERING

Planning and Design

Fourth Edition

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$1 ft = 0.3048\ m$

$1 in = 0.0254\ m$

$$R_{min} = \frac{V_{max}^2}{g(e_{max} + f_s)}$$

Rail: $\frac{F}{G} = \frac{V^2}{gR}$

MSSD = $d_r + d_b$

$d_r = Vt$

$$d_b = \frac{V_i^2 - V_f^2}{2g(f + G)}$$

$\to 9.81$

$\to m/s$

$$d = 0.278\,Vt + \frac{V^2}{254\left(\frac{a}{9.81} + G\right)}$$

without:

$$d = 0.278\,Vt + 0.39\frac{V^2}{a}$$

Chapter 12

Design of Roadways, Railways, and Guideway Systems: Location and Route Layout

This chapter focuses on the location and route layout techniques for streets and highways, railways, and transit guideway systems.

12-1. OVERVIEW OF THE PLANNING AND DESIGN PROCESS

The planning and design process usually consists of several sequential steps or phases. Consider, for example, the planning and design of a roadway or a fixed transit link. If we follow the planning process described previously, the need for a new route is established and the basic features of the link are determined (e.g., termini, general location and size or class, level of service, etc).

The next step is to establish the location of the facility. The location normally is selected from among several alternatives situated within a corridor connecting the termini. After a preliminary location has been chosen, the facility may be designed. This involves specifying the precise horizontal and vertical alignment, dimensions, slopes, construction standards, and quantities and types of materials.

12-2. LOCATION OF TRANSPORTATION LINKS

The location of highways, railways, and other land transport links is an extremely important part of the design process. Decisions made during the location selection process not only determine the cost and operational efficiency of the facility but also influence the disbenefits to or negative impact on nearby communities and the environment. In recognition of the importance of this task, many transportation agencies rely on teams of professionals to perform location studies. These professionals usually include engineers, planners, economists, and social, environmental, and ecological specialists.

The location procedure is an iterative one in which an approximate location selected on the basis of available maps, charts, photographs, and data gradually is narrowed and finally fixed as additional information and data become available and more detailed study is done.

Information for the location study comes in large part from two or more surveys that provide information on the topography, soil conditions, water courses, property lines, utilities, land uses, and so on. Such information is gathered by conventional ground survey methods or by remote sensing techniques, most commonly aerial photogrammetry. While the older techniques still are used on projects of medium and small size, the use of photogrammetric surveys on large works is almost universal and is becoming more frequent on smaller projects. The adaptability of photogrammetry to computer operations has enabled substantial savings in time and money over conventional methods.

By means of aerial or ground surveys, topographic maps are prepared that serve as a basis for the selection of a preliminary and final location. Up to the mid-1990s, such maps were drawn using the inch-pound unit system. With that system, preliminary plans were commonly drawn to the scale of 1 in. = 200 ft, and final design plans were drawn to the scale of 1 in. = 100 ft. With metrication, the scales of transportation plans are referred to as ratios. For example, a scale of 1:1000 in the metric system means 1 m on the plan equals to 1000 m in reality. A scale of 1 in. = 100 ft in the conventional U.S. (inch-pound) system is the same as 1⁄12 ft = 100 ft or a ratio scale of 1:1200. A scale of 1:1000 is commonly used for metric transportation plans in the United States.

By a study of the preliminary map and a stereoscopic examination of aerial photographs, the location team can choose and evaluate possible routes on the basis of traffic service, directness, suitability of terrain, adequacy of crossings of streams and other transport routes, and extent of adverse social, environmental, and ecological effects.

Possible alternative alignments usually are plotted on the base map and, from these alignments with preliminary gradelines, the alternatives are compared. Examples of criteria used in selecting a location are shown in Table 12-1.

For the final design and location, additional mapping is usually necessary. The selected preliminary alignment is used as a guide for the strip area to be mapped. It is common

Location of Highway or railway. function of

*

Table 12-1 Examples of Criteria to be Used in Facility Location Decisions

Criteria	Influencing Factors
Construction costs	Functional classification/design type; topography and soil conditions; current land use
User costs	Traffic volume; facility design features (e.g., gradients intersections); operating conditions (e.g., speeds, traffic control systems)
Environmental impacts	Proximity to sensitive areas; design features to mitigate impacts
Social impacts	Isolation or division of neighborhoods; aesthetics of design; fostering of desired development patterns
Acceptance by various interest groups	Government agencies; private associations and firms; neighborhood groups and the general public

practice, especially for large projects, to utilize low-level aerial photogrammetry to produce base maps along the preliminary alignment, typically at a scale of not less than 1:1200 (1 in. = 100 ft) and with a 0.2-m (1- or 2-ft) contour interval.

A coordinate system is used to indicate the location of key points along an alignment. Usually, horizontal and vertical controls are established precisely along a baseline. Distances between baseline control points often are determined by electronic measuring devices (EMDs). Such devices measure distances by precisely measuring the time required for electromagnetic waves to travel the distance being measured.

The process of the design of the final alignment requires great skill and judgment. The alignment is fitted by hand to the topography and land use until the designer is satisfied that no better fitting can be achieved. Most designers use specially scaled templates and a flexible plastic guide called a spline. Once established, the hand-fitted alignment is converted into a defined line by precisely computing tangent and curve lengths, transition spirals, and the location of control points for the alignment.

12-3. A ROAD LOCATION EXAMPLE

A location study for a 9.2 km (5.7-mile) four-lane controlled-access parkway near Atlanta illustrates the wide variety of factors that must be considered in the location selection process. The purposes of the proposed highway were to improve the accessibility to and from eastern Douglas County and south central Fulton County and to link Interstate Route I-20 west of Atlanta with Route 1-85 near the Hartsfield/Atlanta International Airport.

Originally, two alternate locations were considered, designated *A* and *B* in Fig. 12-1. After a public hearing, alternate *D* was developed and evaluated along with the other two alternates and *C*, the no-build alternate. Nine federal, state, and local agencies were asked to comment on the project.

Alternate *B* was chosen as the preferred location. Although this alternate was the most costly, it caused no families to be relocated and caused no significant impact due to noise. Like alternate A, it crossed the Chattahoochee River just southwest of Buzzard Roost Island, at one of the narrowest widths of the river's floodplain. Table 12-2 summarizes the impacts for each of the alternate locations. It will be noted that many of the impacts were common for the three alternate locations.

ADT: avg daily traffic

12-4. GEOMETRIC DESIGN

DHV: design hourly volume

The essential design features of a roadway or guideway are its location and its cross section. In the horizontal plane, the location of points are referenced to a coordinate system in which the positive *y*-axis is north and the positive *x*-axis is east. Positions along the *y*-axis are called latitudes and those along the *x*-axis are called departures. Customarily, points along the route are identified by *stations*, the distance in feet from some reference point, commonly the beginning point for the project. Under the system of conventional U.S. units, one station was equivalent to 100 ft.[1] With the metric system, a stationing concept based on 1 km is used. For example, station 4 + 325.613 indicates a point 325.613 m from kilometer station 4 + 000. The location of points in the vertical plane (or along the *z*-axis) is given as the elevation above mean sea level.

The cross section of a roadway or guideway is described by its dimensions at a right angle to the direction of the alignment, including widths, clearances, slopes, and so on. Typical dimensions of roadway and guideway cross sections are given in Chapter 13. The

[1] For example, station 42 + 00.00 refers to a point 4200.00 ft from the reference point.

Research and experience are also reflected in the highway design standards currently in use.

12-5. DESIGN CONTROLS AND CRITERIA FOR STREETS AND HIGHWAYS

The elements of highway design are influenced by a wide variety of design controls and criteria. Such factors include:

1. Functional classifications of the roadway being designed
2. Traffic volume and composition
3. Design speed
4. Topography
5. Cost and available funds
6. Human sensory capacities of drivers, bikers, and pedestrians
7. Size and performance characteristics of the vehicles that will use the facility
8. Safety considerations
9. Social and environmental concerns

These considerations are not, of course, independent. The functional class of a proposed highway is largely determined by the volume and composition of the traffic to be served. It may also depend on availability of funds and social and environmental considerations. For a given class of highway, the choice of design speed is governed primarily by topography, which in turn is a determinant of facility cost. Once a design speed is chosen, many of the elements of design may be established on the basis of fundamental sensory capabilities of drivers and other users and on the performance characteristics of vehicles.

The design features of a highway influence its capacity and efficiency, its safety performance, and its social acceptability to highway users, owners of abutting property, and the general public.

The principal design criteria for highways are traffic volume, design speed, the physical characteristics of the vehicles, and the proportions of vehicles of various sizes that use the highways. These criteria are discussed in more detail in the following paragraphs.

12-6. THE RELATIONSHIP OF TRAFFIC TO HIGHWAY DESIGN [1]

The major traffic elements that influence highway design are average daily traffic (ADT), design hour volume (DHV), directional distribution *(D)*, percentage of trucks *(T)*, and design speed *(V)*.

As Section 8-14 describes, traffic volume studies normally involve the measurement or estimation of the ADT. Knowledge of ADT is important for many purposes, but it is not appropriate to use in the geometric design of highways because it does not account for the variation in traffic that occurs in the various seasons of the year, days of the week, and hours of the day. Highway engineers commonly use hourly traffic volumes for purposes of design. Considerable thought has been given to determining which hourly traffic volume would be most appropriate for design. If the peak-hour traffic volume of the year were used, the highway would be grossly overdesigned most of the time, and public funds would be wasted. On the other hand, if the average hourly traffic volume were selected for design, the facility would be inadequate, and intolerable congestion would occur many hours of the year.

In selecting an appropriate design hourly traffic volume, it is helpful to consider a curve such as Fig. 12-2, in which the hourly volumes during the year are arranged in descending order of magnitude. In this illustration, which is typical of rural highway traffic, it will be observed that the maximum hourly volume is 25 percent of the ADT. If the traf-

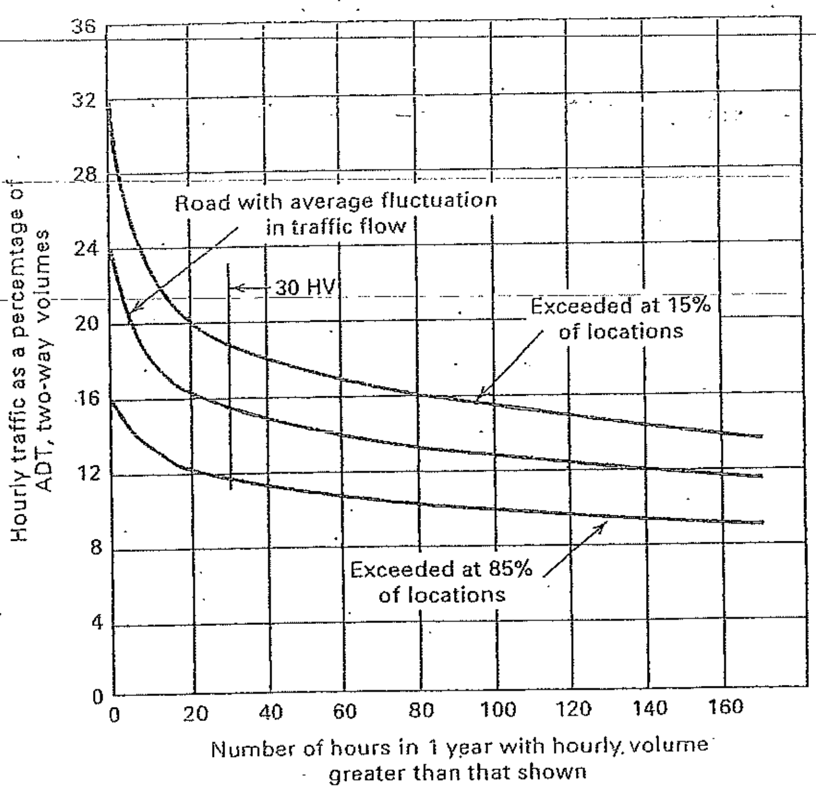


Figure 12-2 Relation between peak-hour and average daily traffic volumes on rural arterials. (Source: *A Policy on Geometric Design of Highways and Streets,* copyright 1994, American Association of State Highway and Transportation Officials. Used by permission.)

fic were uniformly distributed, the maximum hourly volume would be only one-twenty-fourth, or about 4 percent of the ADT. Note that the "knee" of the curve occurs at about the thirtieth highest hourly volume. To the left of this point, the slope of the curve increases sharply; to the right, the curve flattens gradually. The volume corresponding to this point is commonly used as the design hourly volume. It represents a compromise that allows for some degree of congestion during a few hours during the year. Thus, if the thirtieth highest hour is used as the design hourly volume, the designer is willing to tolerate unfavorable operating conditions 29 hr during the year, which is only about 0.3 percent of the total hours during the year.

K Studies have shown that the thirtieth highest hour of traffic expressed as a percentage of ADT does not vary much from year to year, in spite of significant changes in ADT. Thus, it is possible to measure and forecast volumes in terms of ADT and to multiply the ADT by a representative percentage to arrive at a design hourly volume. This percentage K is typically 8 to 12 percent for urban highways, 10 to 15 percent for suburban highways, and 12 to 18 percent for rural facilities.

Directional distribution of traffic is also necessary for geometric design. The directional distribution, D, is the one-way volume in the predominant direction of travel, expressed as a percentage of the two-way design hourly volume. This may average about 67 percent in rural areas and ranges from about 55 near city centers to as much as 70 percent in suburban areas.

Composition of traffic (T) is usually expressed as the percentage of trucks (exclusive of light delivery trucks) during the design hour. That percentage typically varies from about 5 to 10 percent. In urban areas, the percentage of trucks during peak hours tends to be considerably less than percentages on the daily basis.

Table 12-3 Guide for Selection of Design Levels of Service by Type of Area and Appropriate Level of Service[a]

Highway Type	Rural Level	Rural Rolling	Rural Mountainous	Urban and Suburban
Freeway	B	B	C	C
Arterial	B	B	C	C
Collector	C	C	D	D
Local	D	D	D	D

[a] A, free flow, with low volumes and high speeds; B, relatively free flow, but speeds beginning to be restricted by traffic conditions; C, in stable flow zone, but most drivers restricted in freedom to select their own speed; D, approaching unstable flow, drivers have little freedom to maneuver; E, unstable flow, may be short stoppages.

Source: A Policy on Geometric Design of Highways and Streets, American Association of State Highway and Transportation Officials, Washington, DC, 1994.

Service Volumes

It will be recalled from Chapter 8 that the traffic volumes that can be served at each level of service are termed "service volumes." Once a level of service has been chosen for design, the corresponding service volume logically becomes the design service volume. This implies that if the traffic volume using the facility exceeds that value, the operating conditions will be inferior to the level of service for which the roadway was designed. The level of service appropriate for the design of various types of highways is shown by Table 12-3.

Table 8-2 generally describes the operating characteristics of various types of highways operating at each level of service. Such information is useful in determining the number of lanes required to serve the estimated design year traffic. A more extensive capacity analysis, based on data and procedures given in reference 2, may be required to identify potential bottlenecks and to facilitate the design of intersections, ramp terminals, weaving sections, and the like.

Assumed Design Speed. The assumed design speed for a highway, according to the American Association of State Highway and Transportation Officials (ref. [1], p. 62), may be considered as "the maximum safe speed that can be maintained over a specified section of highway when conditions are so favorable that the design features of the highway govern."

The main factor that affects the choice of a design speed is the character of the terrain through which the highway or street is to pass. Thus, an 80-km/hr (50-mph) design speed for a state two-lane arterial might be tolerated in mountainous terrain, while a 110-km/hr (70-mph) design speed might be specified in flat terrain. Choice of design speed should also be logical with respect to the type of highway being designed. In contrast to high-speed arterials, local residential streets should be designed so as to discourage excessive speeds. A design speed as low as 50 km/hr (30 mph), therefore, is recommended for local streets in ordinary terrain, while a lower speed may be specified for hilly terrain. Other considerations determining the selection of an assumed design speed are economic factors based on traffic volume, traffic characteristics, costs of rights-of-way, and other factors that may be of an aesthetic nature. Consideration must also be given to the speed capacity of the motor vehicle. Approved design speeds, as adopted by the American Association of State Highway and Transportation Officials, range from 30 to 120 km/hr with increments of 10 km/hr.

The design speed chosen is not necessarily the speed attained when the facility is constructed. Speed of operation will be dependent upon the group characteristics of the drivers under prevailing traffic conditions. Running speeds on a given highway will vary during the day depending upon the traffic volume. The relationship between the design speed and the running speed will also vary.

Design Designation. The design designation indicates the major controls for which a highway is designed. As already discussed, these include in a broad sense traffic volume, character or composition of traffic, and design speed.

Where access is fully or partially controlled, this should be shown in the design designation; otherwise, no control of access is assumed.

An example of such designations is given below; the tabulation on the left is for a two-lane highway; the tabulation on the right is for a multilane highway.

	Control of access = full
ADT (1997) = 2500	ADT (1997) = 10,200
ADT (2017) = 5200	ADT (2017) = 22,000
DHV = 720	DHV = 2950
D = 65%	D = 60%
T = 12%	T = 8%
V = 110 km/hr	V = 120 km/hr

Vehicle Design. The dimensions of the motor vehicle also influence design practice. The width of the vehicle naturally affects the width of the traffic lane; length has a bearing on roadway capacity and affects the turning radius; the height of the vehicle affects the clearance of the various structures. Weight affects the structural design of the roadway. Vehicle weights, dimensions, and operating characteristics are given in Chapter 4.

12-7. THE NATURE OF RAILROAD AND TRANSIT GUIDEWAY TRACK DESIGN

In the United States, the major system of railways has been completed, for all practical purposes. With the exception of new passenger rail lines such as those planned or being constructed in Atlanta and Miami, the construction of extensive mileage of rail trackage is no longer needed.

In other parts of the world, railroad development continues, including high-speed passenger lines providing speeds up to 270 km/hr (168 mph). A notable example of this type of development is the Train à Grande Vitesse (TGV), a high-speed train constructed to improve railway service from Paris to southeastern France (see Fig. 12-3). Similar systems have been built in Japan, and studies have been conducted in the United States to examine the feasibility of high-speed rail passenger lines in several high-density corridors.

With the freight-hauling railroads, the emphasis is on such activities as the construction of spur tracks to new industrial sites and newly developed mines and power plants. As railroad companies merge, short connecting lines are being constructed to improve movements over the new system. Considerable emphasis is also being placed on improving geometric design features and the riding quality of existing tracks.

From the mid-1960s onward, several financially troubled railroad companies have found it necessary to defer needed track maintenance. These companies, with the encouragement and support of the federal government, are searching for ways to upgrade the tracks in order to make them suitable for heavier loads and higher speeds. The rapid increase in axle loads, especially in the United States, where 115-metric-ton (125-ton) cars are being introduced into service, has led to excessive rail wear and an increase in rail failures. Derailments have increased sharply, many caused by track-related deficiencies

Figure 12-3 The TGV (Train à Grande Vitesse) provides high-speed passenger service in France. The 1153-km system connects most of France's major cities, serving trains that reach speeds of 220 km/hr (137 mph). Each trainset consists of two electrically driven power units and eight articulated trailers with a first class seating capacity of 111 and a second class capacity of 275. The train can be coupled in multiple trainset units. (Courtesy Société National des Chemins de fer Français (SNCF), Centre Audio Visuel (CAV), Photo by Jean-Marc Fabbro.)

such as defective rails and errors in track geometry. After passage of the Staggers Act in 1980, the railroads were able to begin the reconstruction and renovation of tracks, although much remains to be accomplished.

As with highways, the design of new rail lines is influenced strongly by such factors as topography and man-made developments. The nature of the traffic, whether it be freight or passenger, is of great importance. The factors of train operating speeds, freight tonnages, and type of rolling stock also significantly control the elements of geometric design for new railroad lines.

12-8. HIGHWAY AND RAILWAY ALIGNMENT

An ideal and most interesting roadway is one that generally follows the existing natural topography of the country. This is the most economical to construct, but there are certain aspects of design that must be adhered to that may prevent the designer from following this undulating surface without making certain adjustments in a vertical and horizontal direction.

The designer should produce an alignment in which conditions are consistent. Sudden changes in alignment should be avoided as much as possible. For example, long tangents should be connected with long sweeping curves, and short sharp curves should not be interspersed with long curves of small curvature. The ideal location is one with consistent alignment where both grade and curvature receive consideration and satisfy limiting criteria. The final alignment will be that in which the best balance between grade and curvature is achieved.

Terrain has considerable influence on the final choice of alignment. Generally, the topography of an area is fitted into one of the following three classifications: level, rolling, or mountainous.

In level country, the alignment, in general, is limited by considerations other than grade, that is, cost of right-of-way, land use, waterways requiring expensive bridging, existing roads, railroads, canals and power lines, and subgrade conditions or the availability of suitable borrow.

In rolling country, grade and curvature must be considered carefully. Depths of cut and heights of fill, drainage structures, and number of bridges will depend on whether the route follows the ridges, the valleys, or a cross-drainage alignment.

In mountainous country, grades provide the greatest problem, and in general the horizontal alignment (curvature) is conditioned by maximum grade criteria.

12-9. CIRCULAR CURVES

Historically, circular curves have been described by giving either the radius or degree of curve. In highway design, degree of curve is defined as the central angle subtended by a 100-ft arc. This is known as the *arc definition*. The circumference of a circle bears the same relationship to 360° as a 100-ft arc does to the degree of curve D. Therefore,

$$\frac{2\pi R}{360} = \frac{100\text{ ft}}{D} \qquad \left\{D = \frac{5729.58}{R_{\text{ft}}}\right\} \quad \text{HwY} \tag{12-1}$$

Historic railroad practice defined the degree of curve as the central angle subtended by a 100-ft chord. This *chord definition* of degree of curve leads to a different equation:

$$\sin \tfrac{1}{2}D = \frac{50}{R} \quad \text{ft} \qquad \text{RR} \tag{12-2}$$

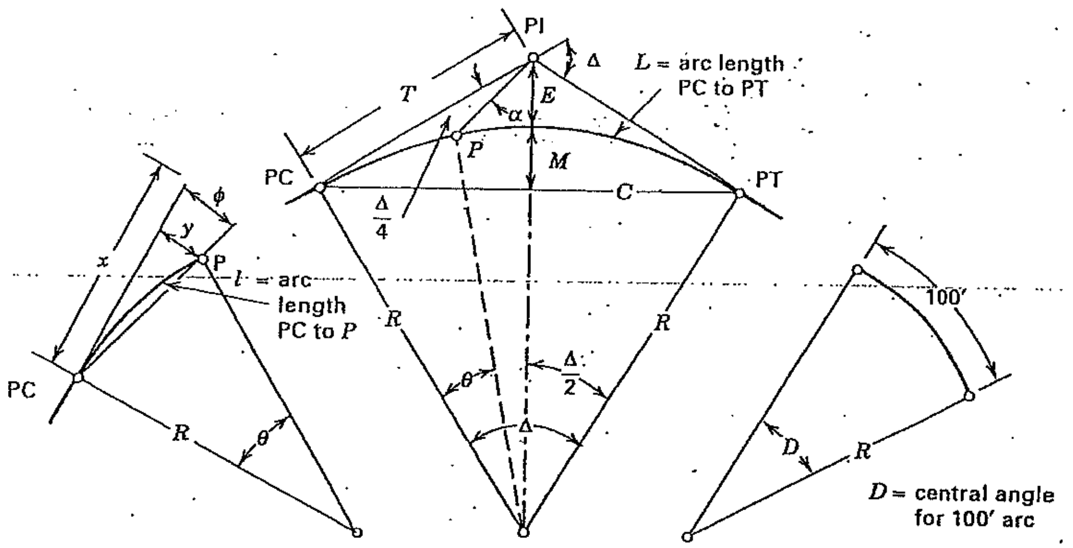
Arc or chord measurements can be considered alike for all curves less than 4° without appreciable error. An examination of tables will show that the following chords may be assumed to be equal to the arcs without noticeable error.

100-ft chords up to 4°
50-ft chords up to 10°
25-ft chords up to 25°
10-ft chords up to 100°

With the metric system, circular curves are defined in terms of the length of radius R in meters rather than by degree of curve. Other important variables include the intersection angle or central angle Δ, the tangent distance T, and the length of curve L. Figure 12-4 illustrates a simple circular curve with variable definitions and basic equations. These equations can be used for either the traditional U.S. units or the metric system.

EXAMPLE 12-1

Given a highway circular curve with an intersection angle $\Delta = 12°30'$, radius of curve = 580.000 m, and a PI (Point of Intersection) station = 9 + 255.628, calculate the station of the point of tangency.



VARIABLES

PC = Point of curvature (Beginning of curve)
PT = Point of tangency (End of curve)
PI = Point of intersection
Δ = Central angle
L = Length of curve (PC to PT)
~~l = Length of arc (PC to P)~~
θ = Central angle for arc length l
T = Tangent length (PC to PI & PT to PI)
φ = Deflection angle at PC between tangent and chord for P
~~α = Deflection angle at PI between tangent and line from PI to P~~
x = Tangent distance from PC to P
y = Tangent offset P

PI = Point of intersection
D = Degree of curvature *(1)*
R = Radius of curve
E = External distance
M = Middle ordinate
~~C = Chord length~~

Each 20m = 1 sta.
100 ft = 1 sta.

CIRCULAR CURVE EQUATIONS

$$D = \frac{5729.57795}{R(ft)} \quad \text{(arc def.)}$$

$$D = 1718.873/R(m)$$

$$L = \frac{2\pi R\Delta}{360} = \frac{\pi}{180} R\Delta$$

$$l = \frac{2\pi R\theta}{360}$$

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

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

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

$$C = 2R\sin\frac{\Delta}{2}$$

$$\phi = \frac{\theta}{2} = \frac{lD}{200}$$

For any tangent distance x,

$$y = R - [R^2 - x^2]^{1/2}$$

For any arc length l,

$$x = R\sin\theta$$

$$y = R(1 - \cos\theta)$$

Note:
(1) This variable used only for curve definition in traditional US units.

Figure 12-4 Properties of a simple circular curve.

SOLUTION

$$\text{Tangent distance } T = R \tan \tfrac{1}{2}\Delta$$
$$= 580.000 \tan(6.25°) = 63.250 \text{ m}$$

$$\text{PC (Point of Curvature) station} = \text{PI station} - T = 9 + 255.628 - 63.520$$
$$= 9 + 128.588$$

$$\text{Curve length } L = 2\pi R\Delta/360° = 2\pi(580.000)\,(12.5°)/360°$$
$$= 126.536$$

$$\text{PT (Point of Tangency) station} = \text{PC station} + L = 9 + 128.588 + 126.536$$
$$= 9 + 255.124$$

12-10. HORIZONTAL ALIGNMENT DESIGN CRITERIA FOR STREETS AND HIGHWAYS

In general, the minimum desirable radius of curve for rural primary and secondary highways is from 350 to 250 m (1148 to 820 ft), which equates approximately to a similar range of curvature of 5° to 7°. Sharper curves with radii as short as 175 m (573 ft, or 10°) may be found in mountainous areas. Many states limit curvature to about 582 m (1910 ft, or 3°) on principal highways. In contrast, a minimum centerline radius of 107 m (350 ft) is permitted for collector streets in ordinary terrain, while local residential streets in hilly terrain may have a centerline radius as short as 34 m (110 ft).

Several variations of the circular curve deserve consideration when developing the horizontal alignment for a highway design. An alignment with two curves in the same direction connected by a short tangent is known as a "broken back" curve. (See Fig. 12-5*a*.) Such an alignment is poor engineering practice. Highway engineers usually prefer a single curve of constant radius or, if conditions require, a compound curve with continuous superelevation, as shown in Fig. 12-5*b*. Where rugged topography or restricted right-of-way necessitate the use of compound curves, the radius of the flatter curve R_L should be not more than 50 percent greater than the radius of the sharper curve, R_S, that is, $R_L < 1.5R_S$.

Another variation of the circular-curve alignment involves the use of reverse curves that are adjacent curves that curve in opposite directions. The alignment illustrated in Fig. 12-5*c*, which shows a point of reverse curvature (PRC), with no tangent separating the curves, would be suitable only for low-speed roads such as those on local streets or in

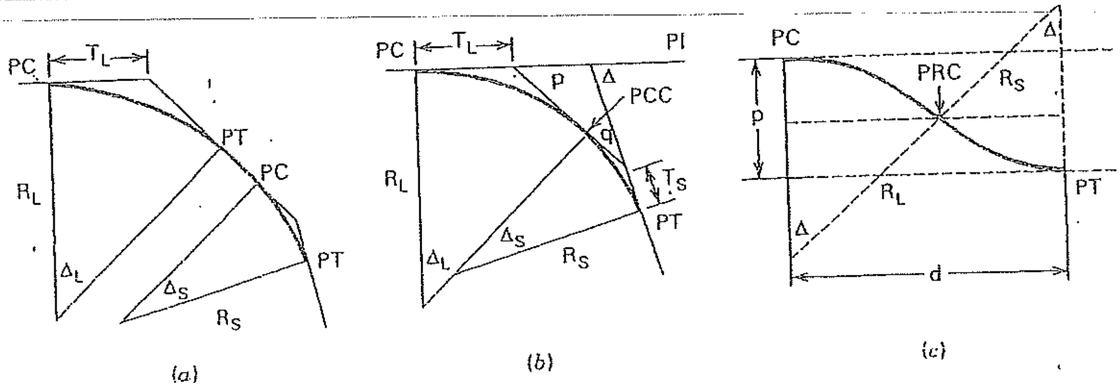


Figure 12-5 Highway alignments. (*a*) Broken-back curve. (*b*) Compound curve. (*c*) Reverse curves.

mountainous terrain. When reversing curves are superelevated, they should be spiraled or separated by a tangent long enough to allow removal of the superelevation of the first curve and attainment of the adverse superelevation for the second curve. (See discussion of attainment of superelevation in Section 12-16). Typically, a separation distance of at least 100 m (328 ft) is required to allow proper transition from one curve to another.

12-11. HORIZONTAL ALIGNMENT DESIGN CRITERIA FOR RAILWAYS AND GUIDEWAYS

Because rail- and guideway vehicles cannot shift laterally, main line tracks and guideways cannot be designed with sudden changes in horizontal alignment. Horizontal curvature limits the speed of rail vehicles and increases the risk of derailments and overturning accidents. Tracks on all but the gentlest curves also require greater horizontal clearances and may create difficulties in coupling. These undesirable effects of curvature stem from the necessity to design for a vehicle unit (the passenger or freight car) that is typically ±26 m (85 ft) long and measures ±18 m (60 ft) between truck centers.

The operation of trains around curves causes wear on the rails and increases train resistance. Curve resistance increases in direct proportion to the degree of curve and amounts to about 0.8 pounds per ton per degree of curve [3]. It has been estimated that a 12° curve approximately doubles the train resistance likely to be experienced on a straight level track [4]. V/ # of vehicles passing a transvers line on highway during a specified time veh/hr/lane
dens/ in specify segment of roadway at a instant veh/h/ln/km

When horizontal curvature is imposed on a section that has steep gradient, it may be necessary to compensate for the curvature by lessening the grade. Since a 1 percent grade results in a resistance of 20 lb/ton, a decrease in grade of 0.8/20 or 0.04 percent will compensate for the resistance of a 1° horizontal curve [3].

Generally speaking, 1° to 3° railroad curves are considered relatively flat curves while 8° to 10° curves are considered relatively sharp. Curves sharper than about 10° are seldom used for main railroad lines, although curves of 16° or even 24° have been utilized in mountainous areas, or on low-speed approaches to terminals in urban centers. Curves as sharp as 40°, however, have been used in railroad yards.

France's TGV high-speed railroad is designed with a normal minimum radius of 4000 m (13,120 ft, or less than 0.5°); however, the design allows for a minimum radius of 3200 m (10,496 ft) for exceptional conditions [5].

Minimum curve radii for urban passenger systems are exemplified by the data in Table 12-4. The differences in the criteria are explained in part by variations in the size of car.

Table 12-4 Minimum Recommended Curve Radii for Urban Passenger Systems

System/Criteria Source	Main Lines m (ft)	Yards and Secondary Tracks m (ft)
Metropolitan Atlanta Rapid Transit Authority (MARTA)	229 (750)[a]	107 (350)
Montreal Bureau de Transport Metropolitain	140 (459)	52 (170)
Italian Transport Organization (UNIFER)	150 (492)	75 (246)

[a]Minimum desirable = approximately 300 m (1000 ft).

Sources: MARTA System Design Criteria, Vol. 1, prepared by Parsons, Brinckerhoff, Quade and Douglas, Inc./Tudor Engineering Company, rev. March 2, 1977; and correspondence with M. Pierre-Paul Arbic, Superintendent of Projects, Montréal Bureau de Transport Metropolitain, September 20, 1979.

For example, the 22.4-m- (75-ft) long car used for Atlanta's system requires a longer minimum radius of curvature than does Montreal's system, which has a 160-m (52.5-ft) car.

12-12. SUPERELEVATION OF HIGHWAY CURVES

On rural highways, most drivers adopt a more or less uniform speed when traffic conditions will permit them to do so. When making a transition from a tangent section to a curved section, if the sections are not designed properly, the vehicle must be driven at reduced speed for safety as well as for the comfort of the occupants. This is due to the fact that a force is acting on the vehicle that tends to cause an outward skidding away from the center of the curve. Most highways have a slight crowned surface to take care of drainage. It can be readily seen that when these crowns are carried along the curve, the tendency to slip is retarded on the inside of the curve because of the banking effect of the crown. The hazard of slipping is increased on the outside of the curve, however, due to the outward sloping of the crown. In order to overcome this tendency to slip and to maintain average speeds, it is necessary to superelevate the roadway sections, that is, raise the outside edge or bank the curve.

Analysis of the forces acting of the vehicle as it moves around a curve of constant radius results in the following simplified formulas:

$$e + f = \frac{v^2}{gR} = \frac{V^2}{127R} \quad \text{(metric system)} \tag{12-3}$$

$$e + f = \frac{v^2}{gR} = \frac{V^2}{15R} \quad \text{(traditional U.S. units)} \tag{12-4}$$

where

e = superelevation rate, m/m (ft/ft)
f = friction factor
v = velocity, m/sec (ft/sec)
V = velocity, km/hr (mph)
R = radius of curvature, m (ft)
g = acceleration of gravity, 9.8 m/sec^2 (32.2 ft/sec^2)

Research and experience have established limiting values for e and f. Use of the maximum e with a safe f value in the formula permits determination of minimum curve radii for various design speeds. Present design practice suggests a maximum superelevation rate of 0.12 m/m (or foot per foot). Where snow and ice conditions prevail, the maximum superelevation rate should not exceed 0.08 m/m. Some states have adopted a maximum superelevation rate of 0.10; however, other rates may be used on some highway types when local conditions require special treatment.

The limiting value of side friction factor f at which the tires begin to skid may be as high as 0.6 or higher. This upper limit of f is not the value used in Eqs. 12-3 and 12-4. In design, engineers use only a portion of the side friction factor, accounting for the comfort and safety of the vast majority of drivers. Thus, in selecting maximum allowable side friction factors for design, engineers use an f value corresponding to the point at which a driver begins to feel uncomfortable and react instinctively to avoid higher speed. Empirical studies have determined that such f values range from about 0.17 at 30 km/hr (20 mph) to 0.09 at 120 km/hr (75 mph). Values within this range may be noted in Table 12-5.

The AASHTO [1] provides graphs and tables showing recommended design values of superelevation for a wide range of design speed, e_{max}, and radius of curve. Figure 12-6 gives an example of the design graphs.

Table 12-5 Example of Minimum Radius Data for Limiting Values of e and f: Rural Highways and High-Speed Urban Streets

Design Speed (km/hr)	Maximum e	Maximum f	Total $(e+f)$	Calculated Radius (m)	Rounded Radius (m)
120	0.06	0.09	0.15	755.9	755
~~30~~	~~0.08~~	~~0.17~~	~~0.25~~	~~28.3~~	~~30~~
40	0.08	0.17	0.25	50.4	50
50	0.08	0.16	0.24	82.0	80
60	0.08	0.15	0.23	123.2	125
70	0.08	0.14	0.22	175.4	175
80	0.08	0.14	0.22	229.1	230
90	0.08	0.13	0.21	303.7	305
100	0.08	0.12	0.20	393.7	395
110	0.08	0.11	0.19	501.5	500
120	0.06	0.09	0.17	667.0	665
30	0.10	0.17	0.27	26.2	25
40	0.10	0.17	0.27	46.7	45
50	0.10	0.16	0.26	75.7	75
60	0.10	0.15	0.25	113.4	115
70	0.10	0.14	0.24	160.8	160
80	0.10	0.14	0.24	210.0	210
90	0.10	0.13	0.23	277.3	275
~~100~~	~~0.10~~	~~0.12~~	~~0.22~~	~~357.9~~	~~360~~
110	0.10	0.11	0.21	453.7	455
120	0.10	0.09	0.19	596.8	595
30	0.12	0.17	0.29	24.4	25
~~40~~	~~0.12~~	~~0.17~~	~~0.29~~	~~43.4~~	~~45~~
50	0.12	0.16	0.28	70.3	70
60	0.12	0.15	0.27	105.0	105
70	0.12	0.14	0.26	148.4	150
80	0.12	0.14	0.26	193.8	195
90	0.12	0.13	0.25	255.1	255
100	0.12	0.12	0.24	328.1	330
110	0.12	0.11	0.23	414.2	415
120	0.12	0.09	0.21	539.9	540

Source: A Policy on Geometric Design of Highways and Streets, American Association of State Highway and Transportation Officials, Washington, DC, 1994.

EXAMPLE 12-2 CALCULATION OF SUPERELEVATION RATES

Calculate the superelevation rates for a roadway with a design speed of 90 km/hr that has a wide range of curve radii, that is, R has values of 585, 440, 350, 295, and 250 m. (These values correspond to degrees of curve D of 3, 4, 5, 6, and 7.) Use e_{max} =0.10. Compare the results with those obtained from Fig. 12-6.

SOLUTION

By Table 12-5, use a maximum side friction factor of 0.13.

By Equation 12-3, the following values of e are calculated:

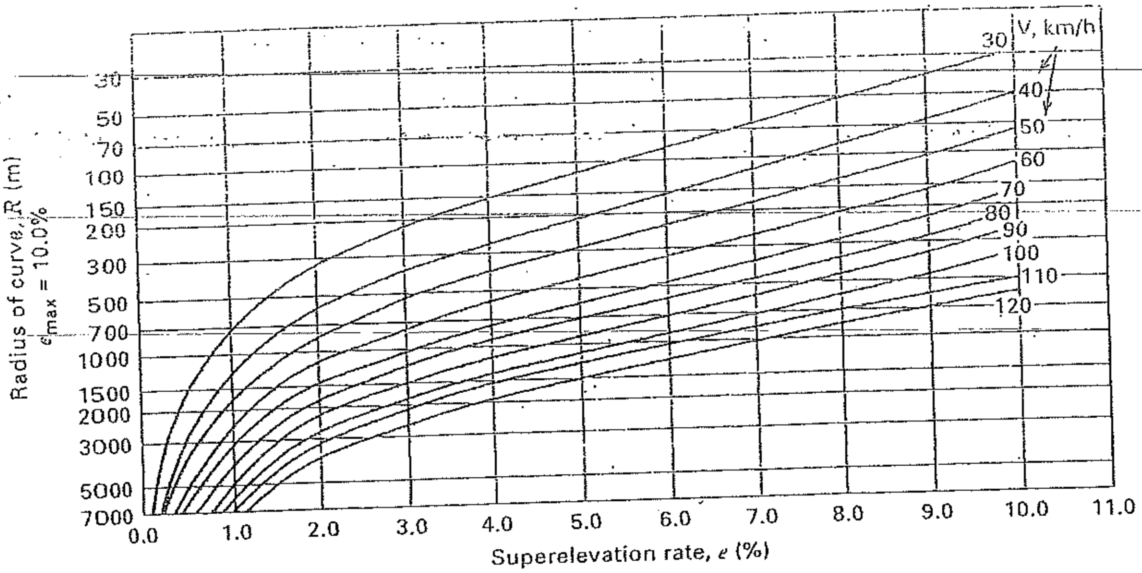


Figure 12-6 Design superelevation rates. $e_{max} = 0.10$. (Source: *A Policy on Geometric Design of Highways and Streets*, copyright 1994, American Association of State Highway and Transportation Officials, Washington, DC. Used by permission.)

R (m)	**Computed value of e**	**Recommended design e, Fig. 12-6**
585	− 0.021	0.070
440	+ 0.015	0.083
350	+ 0.052	0.094
295	+ 0.086	0.099
250	+ 0.125	Exceeds e_{max}

Discussion

For the sharpest curve, the combination of maximum superelevation rate and the maximum side friction factor is insufficient to offset the centrifugal force. This curve is too sharp for the given design speed and maximum superelevation rate and would be unsuitable for the stated conditions.

At the other extreme, for the 585-m curve, a negative value of e was computed. Along this curve, all of the centrifugal force could be offset without exceeding the recommended f_{max} value of 0.13, even with zero superelevation. The AASHTO [1] favors a distribution of superelevation that provides a logical relation between the side friction factor and applied superelevation rate and recommends a positive amount of superelevation for the flattest curve.

The minimum radius of curve for a given design speed can be determined from the maximum rate of superelevation and the side friction factor. The minimum comfortable radius R can be calculated from Eq. 12-3 and 12-4, arranged as follows:

$$R_{min} = \frac{V^2}{127\,(e_{max} + f)} \quad \text{(metric system)} \tag{12-5}$$

$$R_{min} = \frac{V^2}{15\,(e_{max} + f)} \quad \text{(traditional U.S. units)} \tag{12-6}$$

The relationship between superelevation and minimum radius of curve is shown in Table 12-5 for selected metric design speeds. An equivalent table, presented in traditional U.S. units, is included in the appendix.

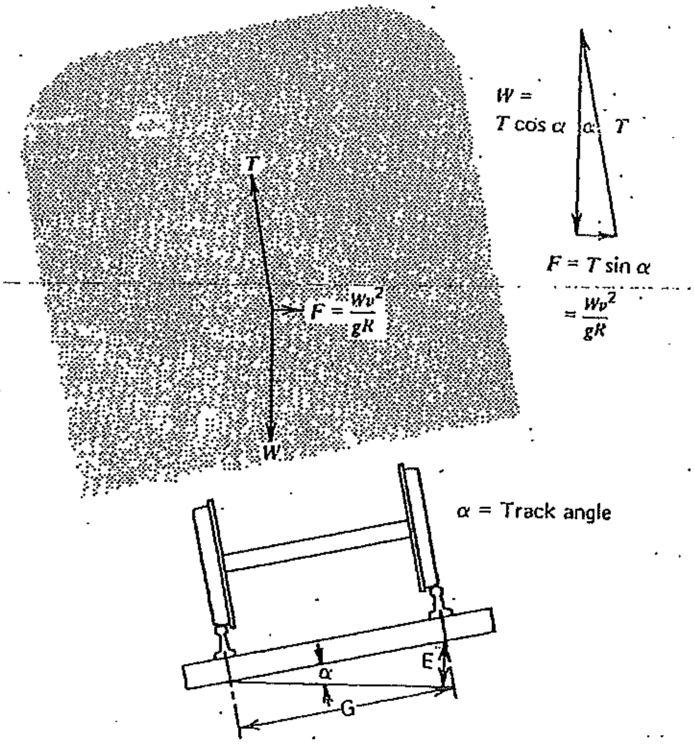


Figure 12-7 Forces on a car body traversing a curve at equilibrium speed. (Source: *Proceedings*, American Railway Engineering Association, Vol. 56, 1955.)

12-13. SUPERELEVATION OF RAILWAY AND TRANSIT GUIDEWAY CURVES

As shown by Fig. 12-7, a train car is subjected to forces of its weight, the resisting forces exerted by the rails, and the centrifugal force:

V = ~~150~~ km/hr
R = 150
Q.s = 1.5

$$F = \frac{mv^2}{R} = \frac{Wv^2}{gR} \quad \text{N (lb)} \qquad \textbf{(12-7)}$$

$\frac{E}{G} = \frac{V^2}{gR}$

E = □ Czer ok!

$\frac{E \mp (UE)}{G} = \frac{V^2}{gR}$

$V_{+75} = \square$

$V_{-75} = \square$

Range

where
m = mass of car, kg (lb)
v = velocity, m/sec (ft/sec)
R = radius of curve, m (ft)
g = acceleration due to gravity, 9.80 m/sec^2 (32.2 ft/sec^2)

A state of equilibrium is said to exist when both wheels bear equally on the rails. Under these conditions, E, the equilibrium elevation, is just sufficient to cause the resultant force, T, to be perpendicular to the plane of the top of the rails. By similar triangles, it can be shown that

$$\cancel{\alpha} \quad \frac{E}{G} = \frac{F}{T}$$

For small track angles where the sine and tangent are approximately equal, the following relationship is essentially correct:

$$\frac{E}{G} = \frac{F}{T} = \frac{v^2}{gR}$$

Using a value of G, the distance between center to center of rails, of 1511 mm (59.5 in.), and expressing train speed in kilometers per hour (miles per hour), the following relationships between the equilibrium elevation, train speed, and radius of curve result[2]:

balanced

$$E = \frac{1511V^2\,(1000/3600)^2}{9.8R} = 11.9V^2/R \quad \text{mm} \quad \text{(metric units)} \qquad (12\text{-}8)$$

For standard

$$E = \frac{11.9V^2}{R} \qquad \sqrt{\frac{E \cdot R}{11.9}}$$

$$E = \frac{59.5V^2\,(5280/3600)^2}{32.2R} = 3.97V^2/R \quad \text{in.} \quad \text{(traditional U.S. units)} \qquad (12\text{-}9)$$

* Emax = 200 mm intercity rail

* Emax = 150 mm urban rail

Eun = 3"–4.5" (75–115) mm on rail

where

V = speed, km/hr (mph)
R = radius of curve, m (ft)

Values of equilibrium elevations for various speeds and degrees of curvature are given in Table 12-6.

It has been found that a rail car will ride comfortably and safely around a curve at a speed that requires an elevation about 3 in.[3] higher than that for equilibrium [3]. It will be noted from Table 12-6A (in the appendix), for example, that for a 60-mph speed and a

Table 12-6 Equilibrium Elevation for Various Speeds on Curves (mm)

Curve Radius (m)	Degree of Curve[a]	48 km/hr (30 mph)	64 km/hr (40 mph)	80 km/hr (50 mph)	96 km/hr (60 mph)	112 km/hr (70 mph)
3943	0° 30'	8	14	22	31	43
1746	1° 00'	16	28	44	63	86
1164	1° 30'	24	42	65	94	128
873	2° 00'	31	56	87	126	171
699	2° 30'	39	70	109	157	214
582	3° 00'	47	84	131	188	256
499	3° 30'	55	98	153	220	
437	4° 00'	63	112	174	251	
349	5° 00'	79	140	218		
291	6° 00'	94	168	262		
250	7° 00'	110	195			
218	8° 00'	126	224			
194	9° 00'	141	251			
175	10° 00'	157	279			
159	11° 00'	172				
146	12° 00'	188				

Emax = Ea + Eun
Emin = Ea − Eun

$$R_{min} = \frac{V^2_{max}}{g(e_{max} + f)}$$

[a]Degree of curve applies to traditional U.S. units.
Source: Equation 12-8.

[2]The historic American Railway Engineering Association (AREA) formula, $E = 0.0007V^2D$, where D = the degree of curve, is more commonly used in U.S. railroads engineering practice.

[3]Modern cars equipped with specially designed suspension systems tend to have less car body roll and can negotiate curves comfortably at 4- or $4\frac{1}{2}$-in. unbalanced elevation [3, 8].

2° curve, the outer rail should be elevated 5.04 in. Experience indicates that a train will ride safely and comfortably around the curve at a speed of about 76 mph, the value corresponding to an elevation of 8.04 in.

Not all trains, of course, travel at the same speed. Even high-speed trains must occasionally slow down or stop on horizontal curves because of traffic interferences or other reasons. Because passengers experience discomfort when the train stops or moves slowly around superelevated curves, a maximum elevation of the outer rail of about 200 mm (8 in.) is recommended and a maximum elevation value of 150 or 175 mm (6 or 7 in.) is desired. For urban passenger rail lines and guideways, a maximum track elevation of about 150 mm (6 in.) is specified [6, 7].

Trains traveling at speeds less than the equilibrium speed tend to cause excessive wear on the inside rail. Similarly, trains that exceed the equilibrium speed cause excessive wear on the outer rail and, in extreme cases, can cause overturning. The velocity that would cause overturning can be computed by Eq. 12-10 or 12-11, which are based on the assumption that the center of gravity of the car is 2.13 m (7.0 ft) above the top of the outer rail:

Overturning speed

error: $V = 6.54R$ (metric units) **(12-10)** $V = 2.24R$ (traditional U.S. units) **(12-11)**

$V = 0.654R$

where

V = speed, km/hr (mph)
R = radius of curve, m (ft)

12-14. SPIRALS OR TRANSITION CURVES FOR HIGHWAYS

Transition curves serve the purpose of providing a gradual change from the tangent section to the circular curve and vice versa. A vehicle that enters a circular curve with transitions travels smoothly and naturally along a curve that gradually changes from a straight line (infinite radius) to a radius of some finite value, R, which is maintained throughout the length of the circular curve. As the vehicle emerges from the circular curve, the curvature is gradually diminished from D to zero. The most commonly used transition curve is the spiral, the radius of which at any point is inversely proportional to its length.

Being unrestrained laterally, automobiles and trucks are free to shift across the traffic lane, allowing the driver to effect artificially a transition from a tangent section (of infinite radius) to a curve section (of finite radius). For this reason, transition curves are not used universally for highway design. Although there is no consistent practice on the use of transition curves by the various highway agencies, spiral transition curves normally are used only on high-volume highways where the degree of curvature exceeds about 3 degrees.

When used in combination with superelevated sections, the superelevation should be attained within the limits of the transition.

The minimum length of the transition curve for highways is given as

$D = \frac{5729.58}{R(ft)}$

$$L_s = 0.035 \frac{V^3}{R} \quad \text{(metric system)} \qquad \textbf{(12-12)}$$

$$L_s = 1.6 \frac{V^3}{R} \quad \text{(traditional U.S. units)} \qquad \textbf{(12-13)}$$

where

L_s = length of transition, m (ft)
V = velocity, km/hr (mph)
R = radius of curvature, m (ft)

Barnett [9] has published tables from which required highway transition curves can be chosen and located without extensive calculations. These tables are still referenced by many road designers.

12-15. SPIRALS OR TRANSITION CURVES FOR RAILWAYS AND TRANSIT GUIDEWAYS

Unlike road vehicles, railroad cars or guideways are restrained by their tracks or guideways and cannot shift laterally or artificially effect a transition in horizontal curvature. For this reason, spiral transition curves are used extensively in mainline railroad design. The use of spiral curves is recommended by the American Railway Engineering Association [3] on all mainline tracks between tangent and curve and between different degrees of curvature where compound curves are used.

According to the AREA [3], the desirable length of the spiral for main tracks where the alignment is being entirely reconstructed or where the cost of the realignment of the existing track will not be excessive should be such that when passenger cars of average roll tendency are to be operated, the rate of change of the unbalanced lateral acceleration acting on a passenger will not exceed 0.03 g/sec. Also, the desirable length in this case needed to limit the possible racking and torsional forces produced should be such that the longitudinal slope of the outer rail with respect to the inner rail will not exceed 1/744, which is based on an 85-ft-long car.

The following two formulas, expressed in traditional U.S. units, are given by the AREA to achieve these results. Equivalent metric equations are also given:

inter ~ uni

$a \leq 0.03\, g/\text{sec}$ comfort

In the metric system

$$L_s = 0.01216 E_u V \quad \textbf{(12-14)} \qquad L_s = 0.744 E_a \quad \textbf{(12-16)}$$

Max

In traditional U.S. units

Max

$$L_s = 1.63 E_u V \quad \textbf{(12-15)} \qquad L_s = 62 E_a \quad \textbf{(12-17)}$$

where

L = desired minimum length of spiral, m (ft)
E_u = unbalanced elevation, $\boxed{\text{mm (in.)}}$ 75 115
E_a = actual elevation, mm (in.)
V = maximum train speed, $\boxed{\text{km/hr}}$ (mph)

$a \leq 0.04\, g/\text{sec}$ realignment

The maximum length of spiral computed by the two formulas should be used.

The AREA [3] recognizes that in the case of realignment of existing tracks, Eq. 12-15 may produce a length of spiral that would be excessively costly to build. In such cases, the Association allows a rate of change of the unbalanced lateral acceleration on a passenger of not more than 0.04 g/sec that results in Eq. 12-19, which may be used in lieu of Eq. 12-15:

$$L = 0.0091 E_u V \quad \text{(metric system)} \quad \textbf{(12-18)} \qquad L = 1.22 E_u V \quad \text{(traditional U.S. units)} \quad \textbf{(12-19)}$$

where

L = desired spiral length, m (ft)
E_u = unbalanced elevation, mm (in.)
V = maximum train speed, km/hr (mph)

Somewhat shorter spiral lengths are specified for construction of conventional urban rail transit systems. For example, the larger of the lengths computed by the following equations is recommended for Atlanta's rail transit system[4]:

Atlanta's rail transit system

In the metric system

$$L = 0.0104E_uV \quad \textbf{(12-20)} \qquad L = 0.72E_a \quad \textbf{(12-22)}$$

In traditional U.S. units

$$L = 1.4E_uV \quad \textbf{(12-21)} \qquad L = 60E_a \quad \textbf{(12-23)}$$

The AREA *Manual for Railway Engineering* [3] lists the various formulas for the calculation of spiral curve data.

12-16 ATTAINMENT OF SUPERELEVATION IN HIGHWAY DESIGN

The transition from the tangent section to a curved superelevated section must be accomplished without any appreciable reduction in speed and in such a manner as to ensure safety and comfort of the vehicle and occupants.

In order to effect this change, it will be readily seen that the normal road cross section will have to be tilted to the superelevated cross section. This tilting usually is accomplished by lowering the inside edge of the pavement and raising the outside edge without changing the centerline grade. This is termed rotating the section about the centerline. Another method is to rotate about the inner edge of the pavement as an axis so that the inner edge retains its normal grade but the centerline grade is varied. Still another method is to rotate the section about the outside edge of the pavement. Rotation about the centerline is used by a majority of the states, but for flat grades this method produces too much sag in the ditch grades. On grades below about 2 percent, rotation about the inside edge of pavement is preferred. Regardless of which method is used, care should be exercised to provide for drainage in the ditch sections of the superelevated areas.

The roadway on full superelevated sections should be a straight inclined section. When a crowned section is rotated to the desired superelevation, the change should be accomplished gradually. This may be done by first gradually elevating the outside edge of pavement until the section from the centerline to the outside edge has a zero slope. The distance to achieve this is called the tangent runout. Second, continue raising the outside edge of the pavement until a uniform cross slope is achieved. Third, rotation about the centerline should be continued until full superelevation is reached.

The length of highway needed to accomplish the change in cross slope from a section with adverse crown removed to a fully superelevated section, or vice versa, is called the *superelevation runoff*. (See Fig. 12-8.) This length is a function of the design speed, full superelevation rate, and pavement width. Recommended minimum lengths of superelevation runoff for two-lane pavements are shown in Table 12-7. On multilane undivided

[4]An even shorter length is permitted where physical restrictions or higher speed requirements make the use of the lengths required by the equations prohibitive or impracticable [6].

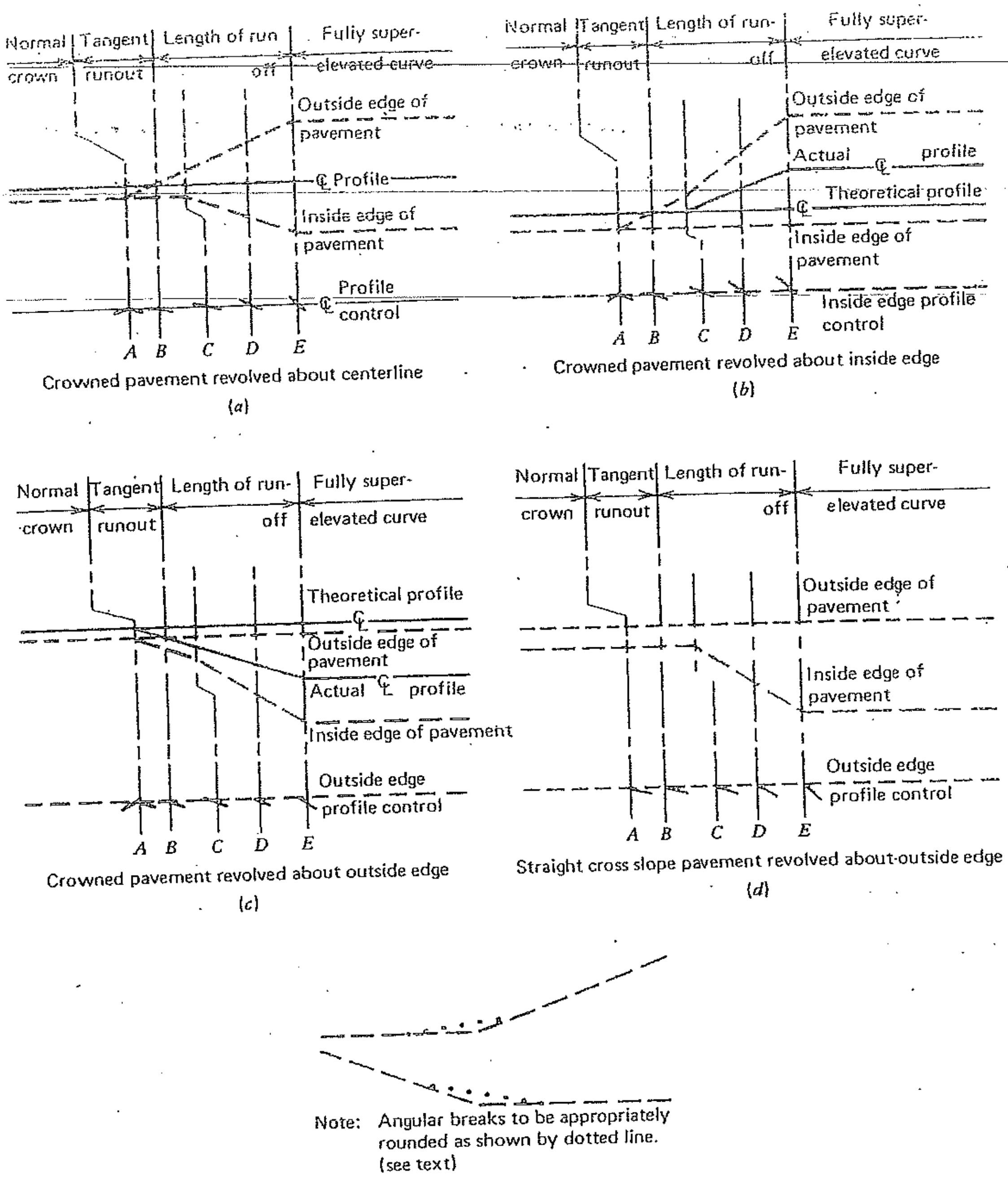


Figure 12-8 Diagrammetric profiles showing methods of attaining superelevation for a highway curve to the right. (Source: *A Policy on Geometric Design of Highways and Streets*, copyright 1994, American Association of State Highway and Transportation Officials, Washington, DC. Used by permission.)

roads or roads with wide medians, increased runoff lengths are necessary, typically 1.2 to 2.0 times the lengths shown in Table 12-7.

Superelevation usually is started on the tangent at some distance before the curve starts and full superelevation generally is reached beyond the point of curvature (PC). In curves of large radius where no spiral transition is used, between 60 and 80 percent of the superelevation runoff is put into the tangent. In curves with spiral transitions the superelevation can be attained within the limits of the spiral.

In order to obtain smooth profiles for the pavement edges, it is recommended that the breaks at cross sections be replaced by smooth curves, as shown in Fig. 12-8. The California Department of Transportation, for example, recommends 30-m (100-ft) long vertical curves for this purpose.

Table 12-7 Length Required for Superelevation Runoff (m): Two-Lane Pavements

Superelevation Rate (%)	Design Speed									
	30 km/hr	40 km/hr	50 km/hr	60 km/hr	70 km/hr	80 km/hr	90 km/hr	100 km/hr	110 km/hr	120 km/hr
					3.6-m Lanes					
2	20	25	30	35	40	50	55	60	65	70
4	20	25	30	35	40	50	55	60	65	70
6	30	35	35	40	40	50	55	60	65	70
8	40	45	45	50	55	60	60	65	70	75
10	50	55	55	60	65	75	75	80	85	90
12	60	65	65	75	80	90	90	95	105	110
					3.0-m Lanes					
2	20	25	30	33	40	50	55	60	65	70
4	20	25	30	35	40	50	55	60	65	70
6	25	30	30	33	40	50	55	60	65	70
8	35	35	40	40	45	50	55	60	65	70
10	40	45	45	50	55	60	65	70	75	75
12	50	55	55	60	65	75	75	80	85	90

Source: A Policy on Geometric Design of Highways and Streets, American Association of State Highway and Transportation Officials, 1994.

12-17. ATTAINMENT OF SUPERELEVATION IN RAILWAY AND TRANSIT GUIDEWAY DESIGN

As in the case of highway curves with spiral transitions, the superelevation of railway curves is attained or run out uniformly over the length of the spiral. This is accomplished by elevating the outer rail. The inner rail is normally maintained at grade.

12-18. GRADES AND GRADE CONTROL

The vertical alignment of the roadway and its effect on the safe and economical operation of the vehicle constitutes one of the most important features of highway and railway design. The vertical alignment, which consists of a series of straight lines connected by vertical parabolic or (outside the United States) circular curves, is known as the gradeline. When the gradeline is increasing from the horizontal, it is known as a plus grade, and when it is decreasing from the horizontal, it is known as a minus grade. In analyzing grades and grade controls, the designer usually studies the effect of change in grade on the centerline profile.

In the establishment of a grade, an ideal situation is one in which the cut is balanced against the fill without a great deal of borrow or an excess of cut to be wasted. All hauls should be downhill if possible and not too long. Ideal grades have long distances between points of intersection, with long vertical curves between grade tangents to provide smooth riding qualities and good visibility. The grade should follow the general terrain and rise and fall in the direction of the existing drainage. In rock cuts and in flat, swampy areas it is necessary to maintain higher grades. Further possible construction and the presence of grade separations and bridge structures also control grades.

Change of grade from plus to minus should be placed in cuts, and changes from a minus grade to a plus grade should be placed in fills. This generally will give a good design, and many times it will avoid the appearance of building hills and producing depressions contrary to the general existing contours of the land. Other considerations for determining the gradeline may be of more importance than the balancing of cuts and fills.

Urban projects usually require a more detailed study of the controls and a fine adjustment of elevations than do rural projects. It is often best to adjust the grade to meet existing conditions because of additional expense of doing otherwise.

12-19. VERTICAL CURVES

The parabolic curve is used almost exclusively in connecting grade tangents because of the convenient manner in which the vertical offsets can be computed. This is true for both highway and railway design. Various configurations of vertical curves are shown in Fig. 12-9. Types and I and II curves are known as crest vertical curves; Types III and IV are called sag vertical curves.

Variable definitions and basic equations for a typical vertical curve are shown in Fig. 12-10. Key elements for a vertical curve are the vertical point of intersection (VPI) at which the gradelines intersect, the vertical point of curvature (VPC) where the curve begins, and the vertical point of tangency (VPT) where the curve ends.

The elevation of a point along the curve is determined by calculating the elevation of the tangent line and then subtracting (in the case of crest curves) or adding (in the case of sag curves) the corresponding offset from the tangent line to the curve. For the

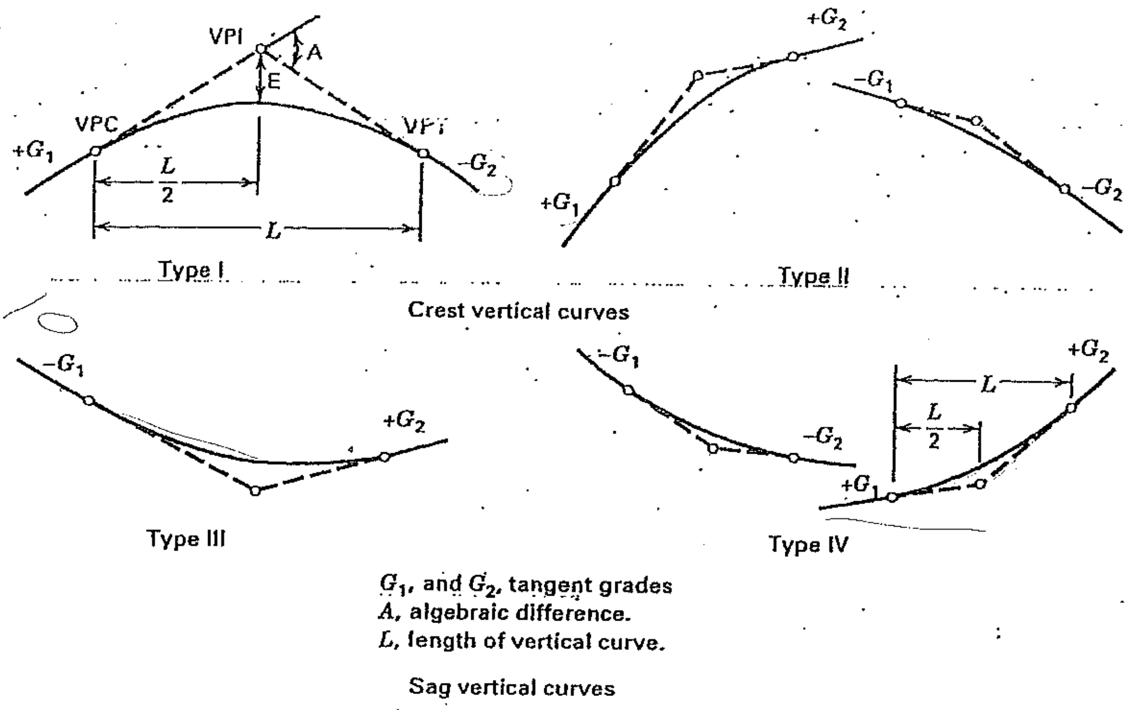


Figure 12-9 Types of crest and sag vertical curves. (Source: *A Policy on Geometric Design of Highways and Streets*, copyright 1994, American Association of State Highway and Transportation Officials, Washington, DC. Used by permission.)

parabola, the offset from the tangent line at the midpoint of the curve or middle ordinate, (E), is equal to

$$E = \frac{AL}{800} = \frac{(g_2 - g_1)L}{800} = \frac{(G_2 - G_1)L}{8} \tag{12-24}$$

where

A = algebraic difference in intersecting grades, percent
L = length of curve, m (ft)
g_1 = grade of the back tangent line, percent
g_2 = grade of the forward tangent line, percent
G_1 = grade of the back tangent line, m/m (ft/ft)
G_2 = grade of the forward tangent line, m/m (ft/ft)

The vertical offset to any point on the curve, y, varies as the square of the distance from the VPC and is expressed as follows:

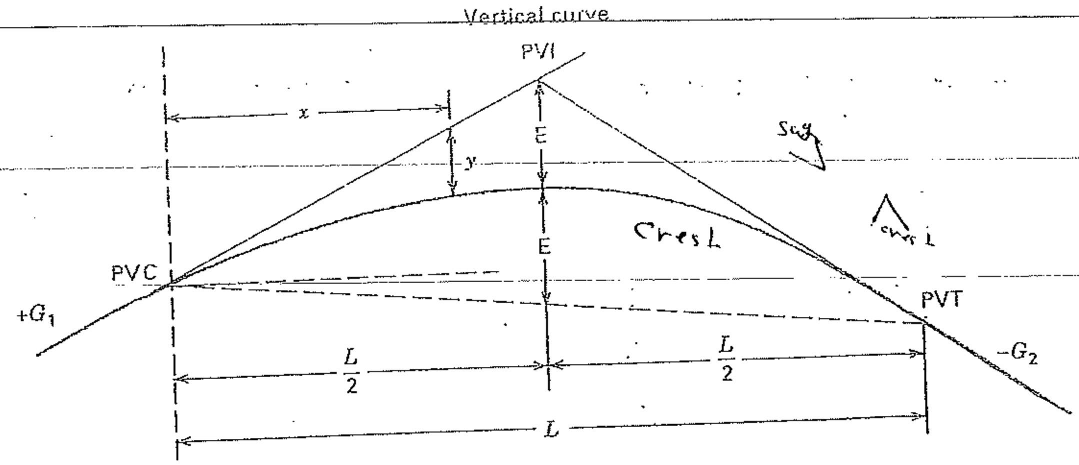
$$y = \left(\frac{x}{L/2}\right)^2 E \tag{12-25}$$

where

x = any distance from the VPC, m (ft)
L = length of curve, m (ft)

Combining Eqs. 12-24 and 12-25, the vertical offset can be expressed as

$$y = \frac{(G_2 - G_1)x^2}{2L} \tag{12-26}$$



Variables

VPI = vertical point of intersection (1)
VPC = vertical point of curvature (1)
VPT = vertical point of tangency (1)
G_1 = grade of initial tangent
G_2 = grade of final tangent
L = length of vertical curve (3)
A = algebraic difference in grade between g_1 and g_2 3-5% H.S.R
x = horizontal distance to point on curve, measured from VPC (3) 2-9% M.R
E_x = elevation of point on curve located at distance x from VPC (2) 16-18% L.R
x_m = location of min/max point on curve, measured from VPC (3)
E_m = elevation of min/max point on curve at distance x_m from VPC (2)
E = external distance = middle ordinate
y = offset of curve from initial grade line

E_{PI} = elevation of VPI (2)
E_{PC} = elevation of VPC (2)
E_{PT} = elevation of VPT (2)
g_1 = grade of initial tangent in percent
g_2 = grade of final tangent in percent

Vertical curve equations

$$A = g_2 - g_1$$

$$E = \frac{(G_1 - G_2)L}{8} = \frac{AL}{800}$$

$$E_x = \underline{E_{PC}} + G_1 x + \frac{(G_2 - G_1)x^2}{2L}$$

$$y = \left(\frac{g_2 - g_1}{2L}\right)x^2 + g_1 x + E_{PVC}$$

For any point p on curve,

$$y = \frac{(G_2 - G_1)x^2}{2L} = \frac{Ax^2}{200L}$$

$$PT_o = \frac{L_s}{2} + (R+S)\tan\Delta/2$$

For high (low) point on curve,

$$x_m = \frac{-g_1 L}{g_2 - g_1} = \frac{g_1 L}{A} \qquad x_m = \frac{-G_1 L}{G_2 - G_1}$$

Notes:
(1) Centerline stations expressed in either meters or feet.
(2) Elevations can be expressed in either meters or feet.
(3) Units for these variables can be expressed in either meters or feet.

Figure 12-10 Properties of a typical vertical curve.

It is usually necessary to calculate elevations at every whole 20-m station or every 50-ft half-station when traditional U.S. units are used. In order to ensure proper drainage or clearances, it is often necessary to compute other critical elevations on the vertical curve such as low points or high points. The low point or high point of a parabolic curve is vertically above or below the vertex of the intersecting grade tangents only if the grades g_1

slope of [illegible]

and g_2 have the same absolute value. Otherwise, the high point or low point will lie to the left or right of the vertex. The distance x_m from the VPC of the curve to the low point or high point is given as

$$x_m = \frac{g_1 L}{g_2 - g_1} \tag{12-27}$$

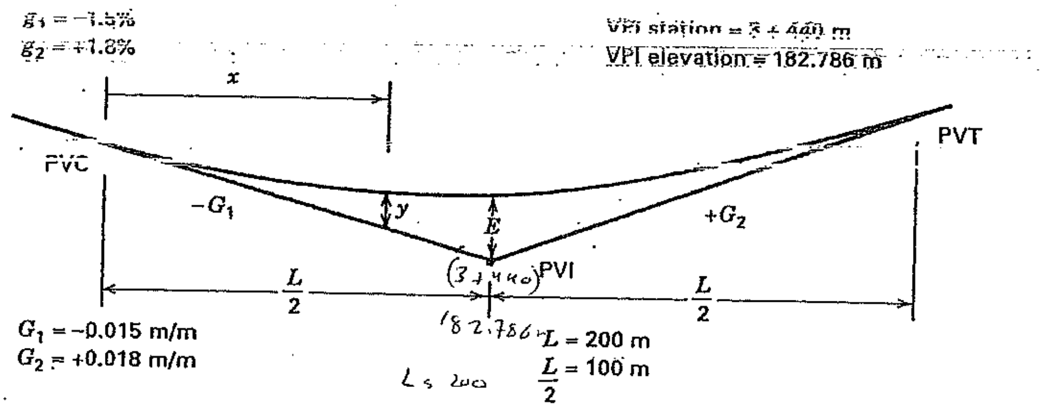
where

g_1 = grade of the back tangent line, percent
g_2 = grade of the forward tangent line, percent
L = length of the curve, m (ft)

The above formulas apply only for equal-tangent vertical curves, which represent most highway vertical curves. Compound parabolic curves, with unequal tangents, are generally warranted only where a symmetrical curve cannot meet imposed vertical alignment conditions.

EXAMPLE 12.3 ***CALCULATIONS OF ELEVATIONS ALONG A SAG VERTICAL CURVE***

A −1.5 percent grade intersects a +1.8 percent grade at station 3 + 440 and elevation of 182.786 m. Given that a 200-m length of curve is utilized, determine the station and elevations of the VPC and VPT. Calculate the elevations of each whole 20-m station and locate the station and elevation of the low point of the curve. A sketch of the given conditions is shown.



Station locations for the VPC and VPT are

VPC station = VPI station − $L/2$ = 3 + 440 − 100 = 3 + 340

VPT station = VPC station + L = 3 + 340 + 200 = 3 + 540

Elevations for the VPC and VPT are

$$E_{VPC} = E_{VPI} + G_1 (L/2) = 182.786 + 0.015\,(100) = 184.286 \text{ m}$$

$$E_{VPT} = E_{VPI} + G_2 (L/2) = 182.786 + 0.018\,(100) = 184.586 \text{ m}$$

Location of the low point can be calculated as follows:

$$x_m = \frac{g_1 L}{g_2 - g_1} = \frac{(-1.5)(200)}{1.8 - (-1.5)} = 90.909 \text{ m (station } 3 + 430.909)$$

Elevation of the low point can be calculated as follows:

$$E_x = E_{\text{VPC}} + G_1 x_m + \frac{(G_2 - G_1)x_m^2}{2L}$$

$$= 184.286 + (-0.015)(90.909) + \frac{(0.018 + 0.015)(90.909)^2}{2(200)}$$

$$= 184.286 - 1.364 + 0.682 = 183.604 \text{ m}$$

Calculations for point elevations at even 20-m stations along the vertical curve can be tabulated as follows:

Station	x **(m)**	**Elevation on Initial Tangent** $(E_{\text{VPC}} + G_1 x)$	y	**Final elevation on curve (elevation on tan** $+ y$**)**
3 + 340	0	184.286	0	184.286
3 + 360	20	183.986	0.033	184.019
3 + 380	40	183.686	0.132	183.818
3 + 400	60	183.386	0.297	183.683
3 + 420	80	183.086	0.528	183.614
3 + 440	100	182.786	0.825	183.611
3 + 460	120	182.486	1.188	183.674
3 + 480	140	182.186	1.617	183.803
3 + 500	160	181.886	2.112	183.998
3 + 520	180	181.586	2.673	184.259
3 + 540	200	181.286	3.300	184.586

12-20. VERTICAL ALIGNMENT DESIGN CRITERIA FOR STREETS AND HIGHWAYS

In the analysis of grade and grade control, one of the most important considerations is the effect of grades upon the operating costs of the motor vehicle. An increase in the gasoline consumption and a reduction of speed are apparent when grades are increased. An economical approach would be to balance the added annual cost of grade reduction against the added annual cost of vehicle operation without grade reduction. An accurate solution to the problem depends on the knowledge of traffic volume and type, which can be obtained only by means of a traffic survey.

While maximum grades vary a great deal in various states, AASHTO recommendations make maximum grades dependent on design speed and topography. Maximum grades of about 5 percent are considered appropriate for a design speed of 110 km/hr (68 mph). For a design speed of 50 km/hr, maximum grades on the more important highways generally are in the range of 7 or 8 percent but could be as steep as 12 percent in rugged topography [1].

Residential streets, which have the primary function of providing access to land areas and only the secondary function of traffic service, tend to conform to and blend with undulations in the existing terrain. Maximum grades as steep as 15 percent may be permitted for local streets in hilly terrain.

Whenever long sustained grades are used, the designer should not substantially exceed the critical length of grade without the provision of climbing lanes for slow-moving vehicles. Critical grade lengths vary with grade percent, the allowable speed reduction, and the operating characteristics of the design vehicle.

Long-sustained grades should be less than the maximum grade used on any particular section of a highway. It is often preferred to break the long-sustained uniform grade by placing steeper grades at the bottom and lightening the grades near the top of the ascent. Dips in the profile grade in which vehicles may be hidden from view should also be avoided.

Minimum grades are governed by drainage conditions. Level grades may be used in fill sections in rural areas when crowned pavements and sloping shoulders can take care of the pavement surface drainage. It is preferred, however, to have a minimum grade of at least 0.5 percent under most conditions in order to secure adequate drainage.

Sight Distance. Safe highways must be designed to give drivers a sufficient distance of clear vision ahead so that they can avoid hitting unexpected obstacles and can pass slower vehicles without danger.

Sight distance is the length of highway visible ahead to the driver of a vehicle. Highway designers are concerned with two types of sight distance. Passing (i.e., overtaking) sight distance is the distance needed for a driver to overtake and pass a slower vehicle on a two-lane highway. Stopping sight distance, which is shorter, is the minimum distance needed to stop a vehicle moving at the design speed of the road short of an unexpected object in the roadway. For safety, a minimum stopping sight distance is required on multilane highways and freeways as well as for two-lane roads.

The minimum stopping sight distance is based upon the sum of two distances: the distance traveled from the time the object is sighted to the instant that the brakes are applied and the distance required for stopping the vehicle after the brakes are applied. The first of these two distances is dependent upon the speed of the vehicle and the perception time and brake reaction time of the operator. The second distance depends upon the speed of the vehicle; condition of brakes, tires, and roadway surface; and the alignment and grade of the highway.

Perception–Reaction Distance. There is a wide variation among vehicle operators as to the time it takes to react and apply the brakes after an obstruction is sighted. As is explained in Chapter 6, the perception–reaction time for alerted drivers averages about $\frac{2}{3}$ sec. For unexpected situations, drivers may require an additional 1 sec or more to react. Even under the simple conditions of research, some drivers have been found to require more than 3.5 sec to recognize the existence of an object and to apply the brakes. For the purpose of computing stopping sight distance, the AASHTO [1] recommends a brake reaction time of 2.5 sec. However, stopping sight distances based on this reaction time may be inadequate when drivers must make complex or instantaneous decisions, when information is difficult to perceive, or when unexpected or unusual maneuvers are required [1]. In such circumstances a longer sight distance called a *decision sight distance*, is recommended. Decision sight distances have been developed by engineers from empirical data and may be found in reference 1. For purposes of ordinary design, the brake reaction time of 2.5 sec is used, and the brake reaction distance is simply the product of 2.5 sec and the speed in meters per second or feet per second.

Braking Distance The approximate braking distance of a vehicle on a level highway is determined by:

$$d = \frac{v^2}{2fg} \tag{12-28}$$

where

d = braking distance, m (ft)
v = velocity of the vehicle when the brakes are applied, m/sec (ft/sec)
f = coefficient of friction between tires and roadway
g = acceleration due to gravity, 9.8 m/sec^2 (32.2 ft/sec^2)

Changing v (in m/sec or ft/sec) to V (in km/hr or mph) and substituting 9.8 m/sec^2 (32.3 ft/sec^2) for g, we have

$d_r = V_i t_r$

$d_b = \frac{V_i^2 - V_f^2}{2g(f \pm G)}$

$d = d_r + \frac{V^2}{254\left(\frac{a}{9.81} \pm G\right)}$

$$d = \frac{V^2}{254f} \quad \text{(metric system)} \tag{12-29}$$

$$d = \frac{V^2}{30f} \quad \text{(traditional U.S. units)} \tag{12-30}$$

It is assumed that the friction force is uniform throughout the braking period. This is not strictly true; it varies as some power of the velocity. Other physical factors affecting the coefficient of friction are the condition and pressure of tires, type and condition of the surface, and climatic conditions such as rain, snow, and ice. Friction factors for skidding are assumed to vary from 0.40 at 30 km/hr (18 mph) to 0.28 at 120 km/hr (75 mph) on wet pavements.

Recommended minimum stopping sight distances on wet pavements are given in Table 12-8. In this table, perception and brake reaction time are combined.

Table 12-8 Stopping Sight Distance on Wet Pavements

Design Speed (km/hr)	Assumed Speed for Condition (km/hr)	Brake Reaction Time (s)	Brake Reaction Distance (m)	Coefficient of Friction, f	Breaking Distance on Level (m)	Stopping Sight Distance for Design (m)
30	30–30	2.5	20.8–20.8	0.40	8.8–8.8	29.6–29.6
40	40–40	2.5	27.8–27.8	0.38	16.6–16.6	44.4–44.4
50	47–50	2.5	32.6–34.7	0.35	24.8–28.1	57.4–62.8
60	55–60	2.5	38.2–41.7	0.33	36.1–42.9	74.3–84.6
70	63–70	2.5	43.7–48.6	0.31	50.4–62.2	94.1–110.8
80	70–80	2.5	48.6–55.5	0.30	64.2–83.9	112.8–139.4
90	77–90	2.5	53.5–62.5	0.30	77.7–106.2	131.2–168.7
100	85–100	2.5	59.0–69.4	0.29	98.0–135.6	157.0–205.0
110	91–110	2.5	63.2–76.4	0.28	116.3–170.0	179.5–246.4
120	98–120	2.5	68.0–83.3	0.28	134.9–202.3	202.9–285.6

Source: A Policy on Geometric Design of Highways and Streets, AASHTO, Washington, DC, 1994.

Effect on Grade on Stopping Distance. When a highway is on a grade, the formula for braking distance is modified as follows:

$$d = \frac{V^2}{254(f + G)} \quad \text{(metric units)} \tag{12-31}$$

$$d = \frac{V^2}{30(f + G)} \quad \text{(traditional U.S. units)} \tag{12-32}$$

in which G is the percentage of grade divided by 100. The safe stopping distances on upgrades are shorter and on downgrades longer than horizontal stopping distances. Where an unusual combination of steep grades and high speed occurs, the minimum stopping sight distance should be adjusted to provide for this factor.

Measuring Stopping Sight Distance. The sight distance available over a crest depends on the fundamental characteristics, specifically the algebraic difference in grades and the length of vertical curve. The basic relationships are as follows

$$\text{When } S < L: \quad L = \frac{AS^2}{100(2h_1 + 2h_2)^2} \tag{12-33}$$

$$\text{When } S > L: \quad L = 2S - \frac{100(2h_1 + 2h_2)^2}{A} \tag{12-34}$$

In determining the stopping sight distance provided by an assumed geometry, it is assumed that the eye height of the average driver is about 1070 mm (3.5 ft) above the pavement. The height of a stationary object in the roadway that the driver must avoid is assumed to be 150 mm (6 in.). With these values and the required stopping distance from Table 12-8, Eqs. 12-33 and 12-34 can be used to calculate the minimum length of the vertical curve for a given algebraic difference in grades and design speed.

Engineers have combined the minimum stopping sight distance required with the sight distance provided by the roadway geometry and developed design controls for vertical curve length. The minimum vertical curve length is expressed in terms of a rate of vertical curvature K in meters (or feet) per percent of A, the algebraic difference in grades. Table 12-9 shows such controls for crest vertical curves, as recommended by the AASHTO [1]. For example, for a design speed (and an assumed travel speed) or 90 km/hr, a rate of vertical curvature $K = 71$ meters per percent of grade is shown. With a plus grade of, say, 4.0 percent intersecting a minus grade of 2.0 percent, the algebraic difference in grades would be 6.0 percent, and the minimum length of the vertical curve would be $6 \times 71 = 426$ m. Note that this length is a minimum length. It may be desirable to use a longer length of curve to balance the earthwork or to provide more favorable operating conditions or a more aesthetically pleasing design [10].

The AASHTO [1] has also published design controls for the minimum length of sag vertical curves. These controls are based on headlight sight distance, rider comfort, drainage control, and the general appearance of the vertical alignment. Recommended minimum K values for sag vertical curves are shown in Table 12-10.

These controls are also shown graphically as Figs. 12-11*a* and 12-11*b*. Note the vertical lines in the lower left of these figures, representing absolute minimum lengths of vertical curves. Most state highway agencies use such absolute minimums when the small

Table 12-9 Design Controls for Crest Vertical Curves

Design Speed (km/hr)	Assumed Speed for Condition (km/hr)	Coefficient of Friction, f	Stopping Sight Distance for Design (m)	Rate of Vertical Curvature, K [length (m) per % of A] Computed	Rounded for Design
30	30–30	0.40	29.6–29.6	21.7–2.17	3–3
40	40–40	0.38	44.4–44.4	4.88–4.88	5–5
50	47–50	0.35	57.4–62.8	8.16–9.76	9–10
60	55–60	0.33	74.3–84.6	13.66–17.72	14–18
70	63–70	0.31	94.1–110.8	21.92–30.39	22–31
80	70–80	0.30	112.8–139.4	31.49–48.10	32–49
90	77–90	0.30	131.2–168.7	42.61–70.44	43–71
100	85–100	0.29	157.0–205.0	61.01–104.02	62–105
110	91–110	0.28	179.5–246.4	79.75–150.28	80–151
120	98–120	0.28	202.9–285.6	101.90–201.90	102–202

Source: A Policy on Geometric Design of Highways and Streets, AASHTO, Washington, DC, 1994. Copyright 1994 by the American Association of State Highway and Transportation Officials. Used by permission.

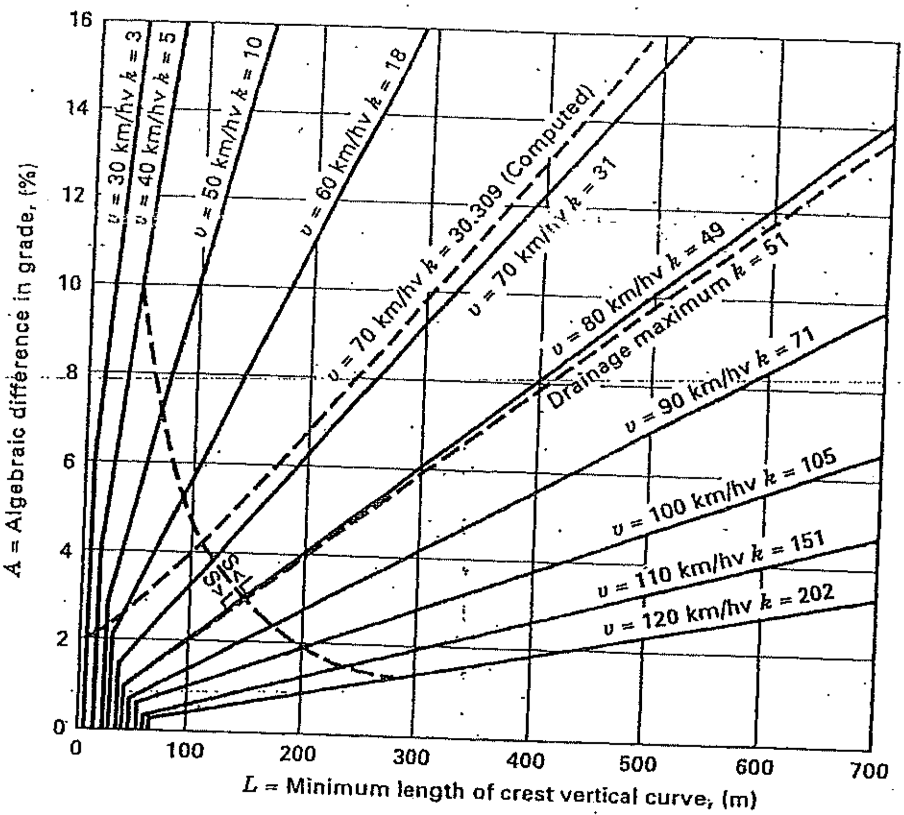
differences in grades and the appropriate k value would indicate an extremely small minimum length of curve. In the figures, the AASHTO uses an absolute minimum length of curve equal to 0.6 times the design speed.

Minimum Passing Sight Distance. A great percentage of U.S. highways carry two lanes of traffic moving in opposite directions. Along such highways, it is necessary to use the lane of opposing traffic in order to pass slower moving vehicles. To overtake the slower vehicle safely, the driver of the passing vehicle must be able to see enough of the highway ahead in the opposing lane to allow sufficient time to pass and then return to the right traffic lane without cutting off the passed vehicle and before meeting the oncoming

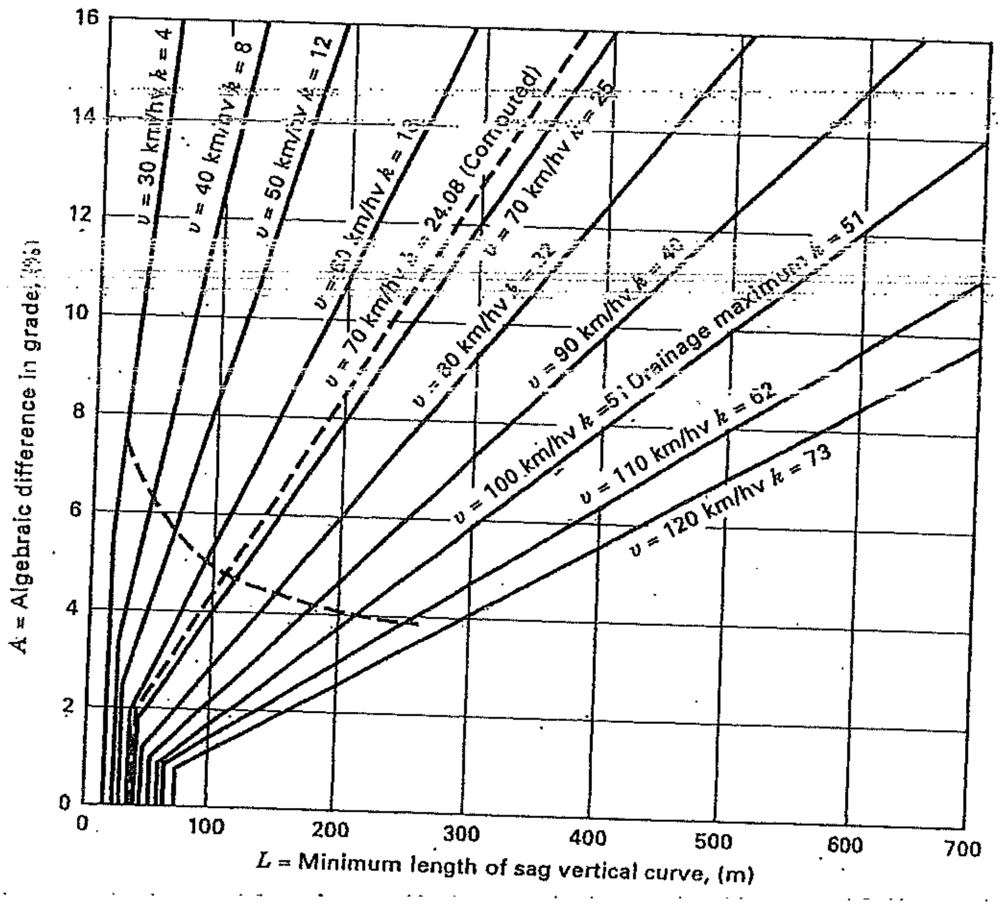
Table 12-10 Design Controls for Sag Vertical Curves

Assumed Design Speed (km/hr)	Speed for Condition (km/hr)	Coefficient of Friction, f	Stopping Sight Distance for Design (m)	Rate of Vertical Curvature, K [length (m) per % of A] Computed	Rounded for Design
30	30–30	0.40	29.6–29.6	3.88–3.88	4–4
40	40–40	0.38	44.4–44.4	7.11–7.11	8–8
50	47–50	0.35	57.4–62.8	10.20–11.54	11–12
60	55–60	0.33	74.3–84.6	14.45–17.12	15–18
70	63–70	0.31	94.1–110.8	19.62–24.08	20–25
80	70–80	0.30	112.8–139.4	24.62–31.86	25–32
90	77–90	0.30	131.2–168.7	29.62–39.95	30–40
100	85–100	0.29	157.0–205.0	36.71–50.06	37–51
110	91–110	0.28	179.5–246.4	42.95–61.68	43–62
120	98–120	0.28	202.9–285.6	49.47–72.72	50–73

Source: A Policy on Geometric Design of Highways and Streets, AASHTO, Washington, DC, 1994. Copyright 1994 by the American Association of State Highway and Transportation Officials. Used by permission.



(a)



(b)

Figure 12-11 Design controls for vertical curves, for stopping distance, upper range. (*a*) Crest vertical curves. (*b*) Sag vertical curves. (Source: *A Policy on Geometric Design of Highways and Streets*, copyright 1994, American Association of State Highway and Transportation Officials, Washington, DC. Used by permission.)

traffic. The total distance required for completing this maneuver is called the passing sight distance.

Empirical research conducted over a period of many years has established sight distances required to provide safe passing maneuvers. The details of how this research was conducted and the assumptions that underlie the design controls for vertical alignment to allow passing are given elsewhere in the literature and will not be repeated here. Recommended minimum passing sight distances for design are given in Table 12-11.

Measuring Passing Sight Distance. In measuring passing sight distance provided by a given vertical alignment, it is assumed that the height of eye of the average driver, h_1, is 1070 mm (3.5 ft) as before. The height of the object, h_2, is the height of an opposing vehicle, taken to be 1300 mm (4.25 ft). With these values and the minimum passing sight distances shown in Table 12-11, Eqs. 12-33 and 12-34 can be used to calculate the minimum length of vertical curve for a given algebraic difference in grades and design speed.

The minimum length of the vertical curve to allow passing can also be calculated from the rates of vertical curvature per percent of algebraic grade, K, shown in Table 12-11.

Sight Distance for Multilane Highways. It is not necessary to provide passing sight distance along streets or highways that have two or more traffic lanes in each direction of travel. Multilane highways should have continuously adequate stopping sign distance, with greater than minimum distances being provided wherever possible.

Horizontal Sight Distances. Horizontal sight distance along the inside of a curve may be limited by obstructions such as hedges, buildings, high ground, or other topographic features. Such obstructions are usually plotted on highway plans, and available horizontal sight distance can be scaled directly from the plans. The scaled

Table 12-11 Design Controls for Crest Vertical Curves Based on Passing Sight Distance

Design Speed (km/hr)	Minimum Passing Sight Distance for Design (m)	Rate of Vertical Curvature, K, Rounded for Design [length (m) per % of A]
30	217	50
40	285	90
50	345	130
60	407	180
70	482	250
80	541	310
90	605	390
100	670	480
110	728	570
120	792	670

Source: A Policy on Geometric Design of Highways and Streets, AASHTO, Washington, DC, 1994. Copyright 1994 by the American Association of State Highway and Transportation Officials. Used by permission.

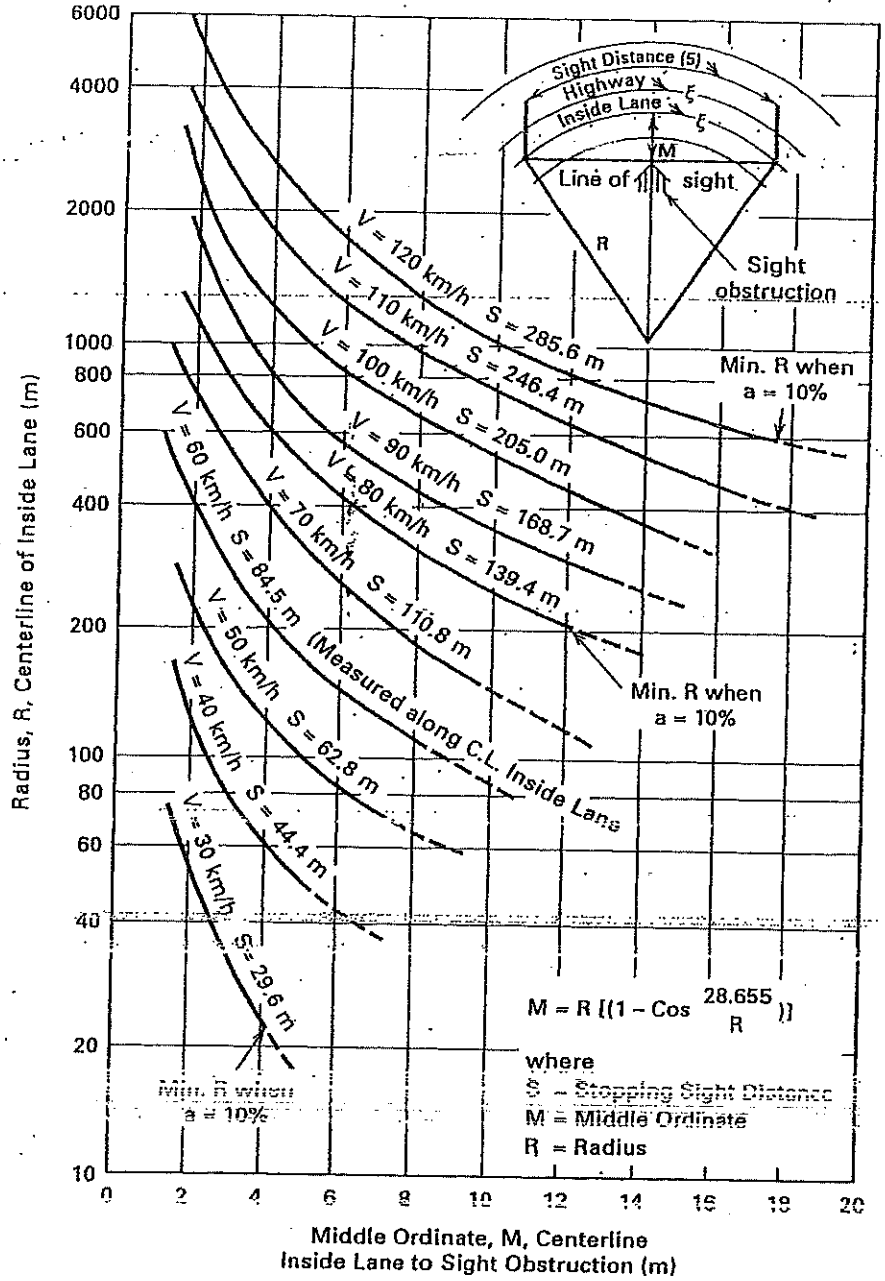


Figure 12-12 Required middle ordinates to satisfy the upper values of stopping sight distances (Table 12-8) for curves of various degrees under open conditions. (Source: *A Policy on Geometric Design of Highways and Streets,* copyright 1994, American Association of State Highway and Transportation Officials, Washington, DC. Used by permission.)

distance can then be compared with the required minimum stopping sight distance shown in Table 12-8.

The relationship between the radius of the centerline of the inside lane and the value of the middle ordinate necessary to provide stopping sight distance on horizontal curves is expressed by

$$M = R\left(1 - \cos\frac{28.65S}{R}\right) \qquad \textbf{(12-35)}$$

where

S = stopping sight distance, m (ft)

~~M = middle ordinate, or distance to obstruction from centerline of inside lane, m (ft)~~
R = radius of centerline of inside lane, m (ft)

Figure 12-12 shows a horizontal sight distance chart developed from Eq. 12-35 and using the upper values of stopping sight distance from Table 12-8.

12-21. VERTICAL ALIGNMENT DESIGN FOR RAILROADS AND TRANSIT GUIDEWAYS

Grade design for railroads is similar in many respects to that for highways. As in highway design, there is a need to provide smooth and consistent vertical alignment and to consider controlling elevations of crossing and connecting railroads, highways, bridges, and drainage structures. Vertical parabolic curves are used to connect intersecting railroad gradelines, and the calculation of curve elevations is accomplished as described in Section 12-19.

Railroad vertical alignment design differs significantly in several respects from the profile grade design of highways. These differences arise from inherent vehicle differences and result in more stringent design criteria for railroads. The need for stricter design criteria for railroads is principally attributed to two considerations:

1. The much longer and heavier railroad vehicle
2. The relatively low coefficient of friction between the driver wheels and the rails

Railroad design is characterized by much smaller maximum grades and much longer vertical curves than are highways. Generally, steep grades cannot be tolerated in railroad design. The maximum grade for most main lines is about 1.0 percent, although grades as high as 2.5 percent may be used in mountainous terrain. This is especially true for railroads that accommodate freight trains. Slightly greater grades can be tolerated for railroads designed exclusively for passenger service. For example, a maximum gradient of

$L = \frac{A}{c}$ $A \rightarrow |g_1 - g_2|$

Interesting

Table 12-12 Vertical Curve Criteria for Railroads

	Maximum Rate of Change of Gradient, Percent per Station[a]	
	In Sags	On Crests
High-speed main tracks	0.05	0.10
Secondary main tracks	0.10	0.20

Metric Units

	Maximum Rate of Change of Gradient, Percent per Meter	
	In Sags	On Crests
High-speed main tracks	0.00164	0.00328
Secondary main tracks	0.00328	0.00656

[a] 1 station = 100 ft.

Source: Adapted from *Manual for Railway Engineering*, Vol 1, American Railway Engineering Association, 1995.

3.0 percent has been specified for Atlanta's conventional rail transit system. France's TGV railroad is designed for a maximum grade of 3.5 percent. In Montreal, where rubber-tired vehicles are used, a maximum gradient of 6.5 percent was used for the guideway.

A minimum gradient of about 0.3 percent may be required in underground and on aerial line structures to accommodate the drainage.

Table 12-12 gives the specifications of the AREA [3] for the calculation of the lengths of vertical curves for mainline railroad tracks. Generally, much shorter curves are permitted along urban rail lines, as Figs. 12-13 and 12-14 demonstrate.

EXAMPLE 12-4

A +0.8 percent grade intersects a −0.3 percent grade on a high-speed main track. What minimum length of vertical curve in feet and meters should be used?

The curve is on a crest. The total change in grade is 1.1 percent.

$\boxed{\Delta t\ 100 = \text{sta } 15}$ 11sta + 100 ft

.8 − (−0.3) = 1.1 = A

$$\text{Length of vertical curve} = \frac{1.1}{0.1} = 11 \text{ stations or } 1100 \text{ ft}$$

L = A/r → algebraic diff in grade (abs value)

$$= \frac{1.1}{0.00328} = 335 \text{ m}$$

Length of vertical curve — change of grade (table 12.11)

EXAMPLE 12-5

A −0.4 percent grade intersects a + 1.2 percent grade on a high-speed main track. What minimum length of vertical curve is required (expressed in feet and meters)?

This curve occurs in a sag. The total change of grade is 1.6 percent:

$$\text{Length of vertical curve} = \frac{1.6}{0.05} = 32 \text{ stations or } 3200 \text{ ft}$$

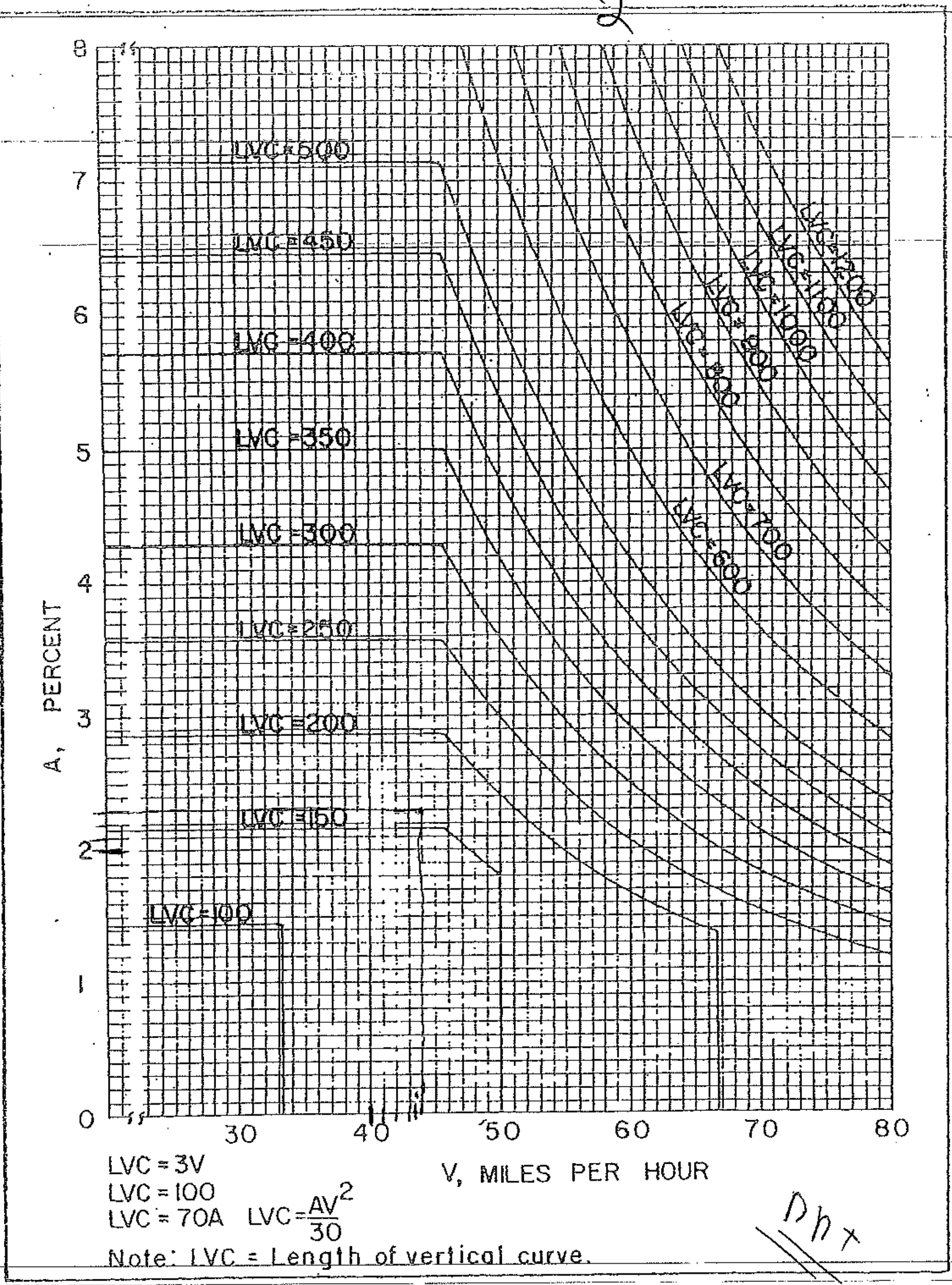
$$= \frac{1.6}{0.00164} = 976 \text{ m}$$

It will be noted that the criterion for length of vertical curve is more critical for sags than for crests. Longer vertical curves are required for sag curves because of the tendency of undesirable slack to develop in the couplings as the cars in the front are slowed down by the change in grade. The subsequent removal of the slack causes jerking to occur, which in extreme cases can cause a long freight train to break in two.

Finally, the suitability of a railroad profile grade design depends on the ability of trains to operate over the line smoothly and economically and whether trains of a given size (tonnage) will stall on the maximum grade.

In studying performance characteristics of a locomotive and train of cars over a particular stretch of track, the construction of a *velocity profile* may be helpful. The velocity profile is an analog of the Bernoulli principle in fluid mechanics (see Section 14-9.) It is based on the fact that the total energy of a moving train is the sum of the elevation head (potential energy) and the velocity head (kinetic energy). An example of a velocity profile is shown in Fig. 12-15. In this figure, the solid line represents the actual profile of the

Sag



Sag

Figure 12-13 Design graph for ~~crest~~ vertical curves for a rail transit line. (Source: *MARTA System Design Criteria,* Vol. 1, prepared by Parsons, Brinckerhoff, Quade and Douglas, Inc./Tudor Engineering Co., rev. March 2, 1977.)

track, the elevation head. The dashed line represents the virtual or velocity profile, the elevation head plus the velocity head.

The velocity head, h, of a moving train is approximately equal to that of a freely falling body:

$$h = \frac{v^2}{2g} \quad \text{(metric system)} \tag{12-36}$$

Crest

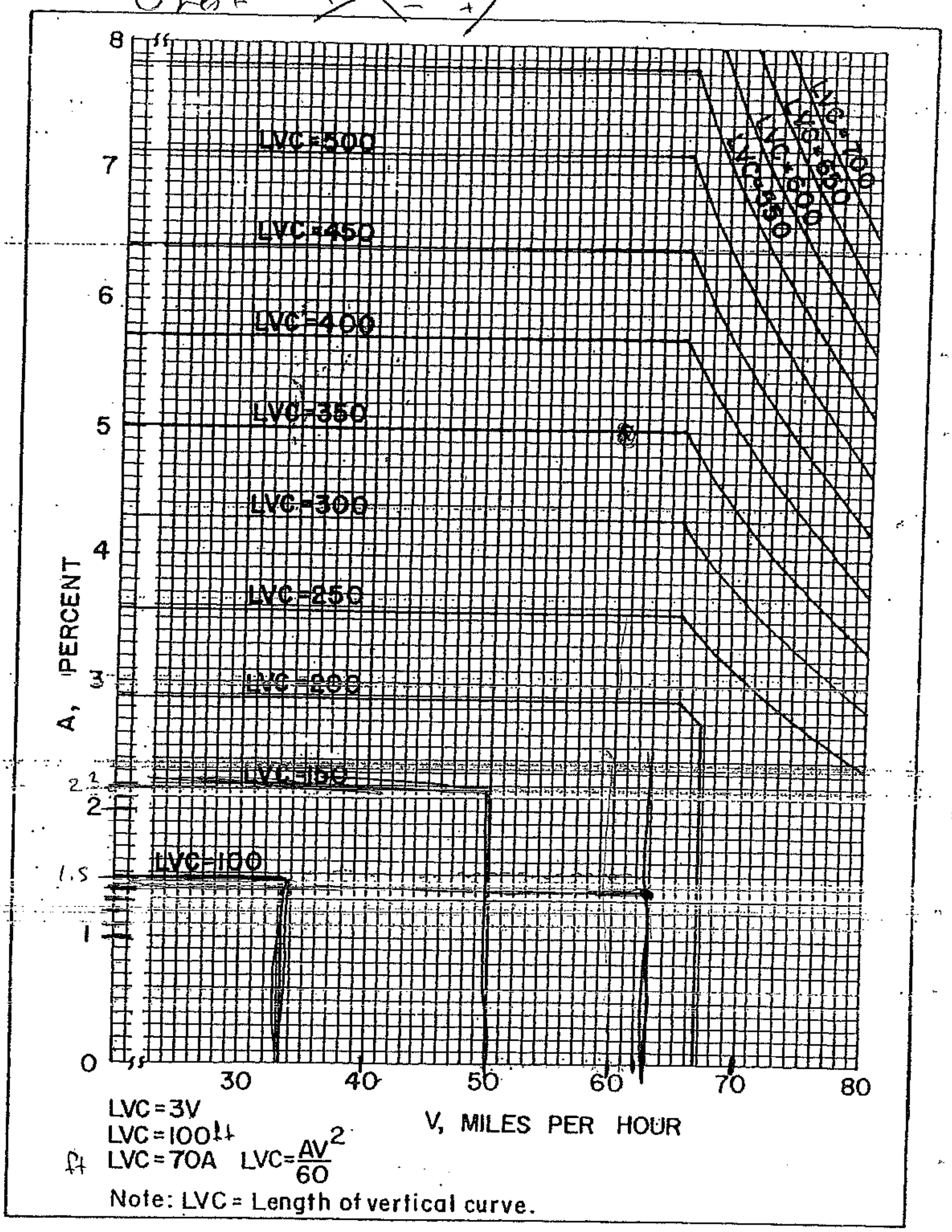


Figure 12-14 Design graph for ~~sag~~ crest vertical curves for a rail transit line. (Source: *MARTA System Design Criteria,* Vol. 1, prepared by Parsons, Brinckerhoff, Quade and Douglas, Inc./Tudor Engineering Co., rev. March 2, 1977.)

$$h = \frac{v^2}{2g} \quad \text{(traditional U.S. units)} \tag{12-37}$$

where

v = train velocity, m/sec (ft/sec)

g = acceleration due to gravity, 9.8 m/sec^2 (ft/sec^2)

1km = 1.6093 mile

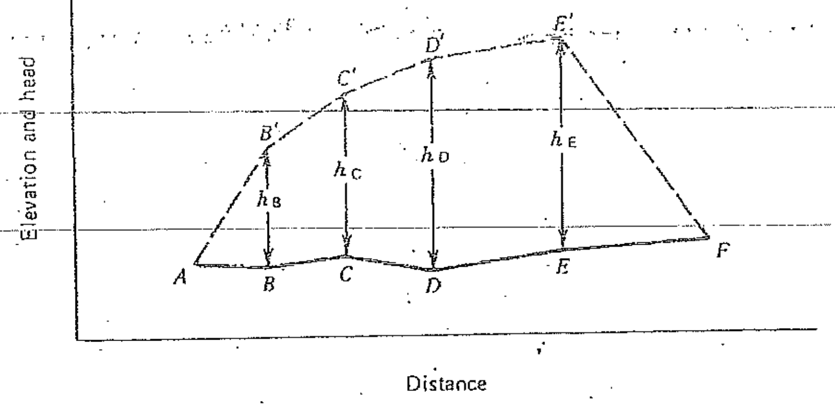


Figure 12-15 A velocity profile.

Crest $g_1 = +1.95$ %, $g_2 = 0.55$ % $A = 1.4$ % $C = 0.00656$ > $LVC = \frac{A}{C}$ (m)

According to the AREA, this value should be increased by about 6 percent to allow for energy stored in the rotating wheels. This results in the equations

$$h = 0.0041V^2 \quad \text{(metric system)} \tag{12-38}$$

$$h = 0.035V^2 \quad \text{(traditional U.S. units)} \tag{12-39}$$

where 100 km/hr ⇒ $100 \frac{km}{hr} \cdot \frac{1 km}{1.6093} = 62.14$ mph

V = train speed, km/hr (mph)

In the figure, line AB' represents the maximum grade the particular locomotive and train of cars can negotiate at a speed corresponding to point B. This grade is called the *acceleration grade*. Acceleration grade is computed by the following steps:

1. Determine the net tractive effort available for acceleration[5] by subtracting the train resistance (on tangent level track) from the drawbar pull of the locomotive.
2. Express the net tractive effort available for acceleration in pounds per ton by dividing by the total weight of the train.
3. Divide the value obtained in step 2 by 98 N/metric ton/percent grade (20 lb/ton/percent grade), the resistance due to grade. The result is the acceleration grade.

It will be noted that acceleration grade decreases with increase in speed because of similar decreases in drawbar pull.

The general procedure for the construction of a velocity profile is to lay out on a graph of the actual profile the equivalent acceleration grades for speed increments of 0 to 10 km/hr, 10 to 20 km/hr, and so on. For example, consider first the 0 to 10 km/hr increment. From the first point on the survey profile (point A), a slope equal to the acceleration grade is laid out until the distance between the two gradelines is equal to the velocity head corresponding to 10 km/hr, $h_B = 0.0041(10)^2 = 0.41$. From point B', a slope equal to the acceleration grade until the distance between the velocity gradeline and the actual gradeline is equal to the velocity head corresponding to 10 mph, $h_c = 0.0041(20)^2 = 1.64$. This procedure is repeated until the maximum speed is reached. When this occurs, the velocity

1.41

[5] This effort may also be used for climbing grades.

$\frac{m}{s}$ $\frac{s}{\cancel{s}}$ $\frac{m/s}{km/se}$

profile is parallel to the actual profile and continues to be parallel until a steeper grade is reached or the locomotive decelerates due to braking or a reduction of the throttle.

Given a freight train of a certain tonnage and the minimum desirable climbing speed, the velocity profile can be used to determine the ruling grade. The ruling grade is defined as the maximum gradient over which a given locomotive pulling maximum tonnage can be hauled at a given constant speed. On a ruling grade, the velocity profile is parallel to the track profile as line $D'E'$ in the figure.

On a grade greater than the ruling grade, termed a momentum grade, a part of the momentum of the locomotive and train is used in ascending the grade, resulting in a reduction in speed. The extent of this reduction can be determined by the velocity profile. On a momentum grade, the train decelerates, and the actual and velocity profiles converge.

An extensive discussion of the velocity profile and its application to railroad vertical alignment design has been given by Hay [4].

PROBLEMS

1. Given an intersection angle $\Delta = 23°42'$ right and a degree of curve $D = 1°45'$, compute the tangent distance, curve length, and station of the point of tangency (PT) of a circular horizontal curve by the chord definition. The station of the point of intersection (PI) is 27 + 85.50.
2. Solve problem 1 in SI (metric) units. The metric station of the PI is 0 + 849.020. Compute the curve data and metric station of the PT.
3. A +1.7 percent grade intersects a −4.3 percent grade at station 150 + 60 at an elevation of 657.88. Calculate the centerline elevation for every even 100-ft station for a 400-ft vertical curve.
4. A −2.3 percent grade intersects a −1.6 percent grade at metric station 6 + 850 and an elevation of 190.587 m. Calculate the centerline elevations in meters at metric stations 6 + 700 and 6 + 900 for a 300-m curve.
5. A −4.8 percent grade intersects a +3.7 grade at station 50 + 00 and elevation 532.20. To provide proper cover for a culvert, it is desired that the elevation of the vertical curve at station 50 + 00 be 539.00 or higher. What minimum length of vertical curve will provide the necessary cover? If a 700-ft curve is used, determine the station and the elevation of the low point of the curve.
6. A vertical parabolic curve is to be used under a railroad grade structure. The curve is 300 m long. The minus grade from left to right is 3.2 percent, and the plus grade is 5.6 percent. The intersection of the two grades is at metric station 4 + 525 and an elevation of 483.571 m. Calculate the station and elevation of the low point of the curve.
7. Determine the equilibrium elevation for a railroad curve with a radius of 600 m given a design speed of 80 km/hr. What would be the overturning speed for these conditions?
8. Determine the equilibrium elevation, in inches, for a 1°45′ railroad curve given a design speed of 60 mph. What would be the overturning speed for these conditions?
9. It is the policy of a railroad to use a 150-mm maximum elevation of the outer rail. Given a 100-km/hr maximum train speed and a curve radius of 550 m, determine the superelevation that should be used and the minimum length of spiral.
10. It is the policy of a certain urban railroad to use a 7-in. maximum elevation of the outer rail. Given a 75-mph maximum speed and a 2.5° of curve, determine the superelevation that should be used and the minimum length of spiral.
11. A −1.65 percent grade intersects a −0.45 percent grade on a high-speed main rail-

~~road track. What minimum length of vertical curve should be used? Express your an~~swer in meters and in feet.

12. A highway curve has been designed according to AASHTO criteria that assume the driver's height of eye to be 1070 mm (3.5 ft) above the pavement. The design speed is 100 km/hr, and the algebraic difference in grades is 3.0 percent. Suppose the automobile manufacturers changed automobile design, causing the average height of eye to be 914 mm (3.0 ft). What would be the implications regarding the design of the curve?
13. A 7.2-m-wide highway has a centerline radius of 450 m. The highway was designed for a speed of 100 km/hr. Determine if an object located $M = 10$ m from the centerline of the inside lane constitutes a hazard.
14. Explain why spiral transition curves are used extensively in railroad design but rarely in highway design.

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Ballast 400

Railway c.s
397

Crossties
402

$$d_r = V_i t_r$$

$$d_b = \frac{V_i^2 - V_f^2}{2g(f \mp G)}$$

$$MRSSD = d_r + d_b$$

$$d = 0.278 V t + \frac{V^2}{254\left(\left(\frac{a}{9.81}\right) \mp G\right)}$$

Chapter 13

Design of Roadway, Railway, and Guideway Systems: Sections and Intersections

This chapter is concerned with two major aspects of the design of roadway, railway, and guideway systems: the establishment of the dimensional features of the cross section and the treatment of the intersection of two or more transport links.

THE ROADWAY CROSS SECTION

13-1. NUMBER OF LANES

The number of lanes should be capable of accommodating the anticipated type and volume of traffic. Roads currently in use include two lane, three lane, multilane undivided, and multilane divided.

Over 90 percent of rural roads in the United States have two lanes. These facilities vary from remote unpaved roads that follow the natural ground surface to heavily traveled, high-speed primary highways with paved surfaces and stabilized shoulders.

Three-lane highways may be used to increase the capacity of two-lane highways by providing for more efficient passing and reducing left-turn conflicts. This is accomplished in several ways:

1. Three-lane highways may be operated permanently over a long segment with two lanes in one direction and one lane in the other.
2. The third lane may be alternately assigned to one direction, then to the other, providing alternating passing lanes for each direction of flow.
3. The third lane may be an auxiliary climbing lane for heavy vehicles in the uphill direction.
4. In suburban areas, the center lane of three-lane highways is sometimes used as a continuous left-turn lane.

It is vitally important that three-lane highways be properly signed and marked; otherwise, they can become very dangerous facilities.

Although the four-lane highway is the basic multilane type, traffic volumes may warrant the use of highways having six or even eight lanes, particularly in urban areas.

The *Highway Capacity Manual* [1] recommends a simple planning analysis to determine the probable number of lanes required and whether a multilane highway will be appropriate for the expected conditions. The planning procedure requires information on:

1. The general terrain through which the highway is to be built
2. The annual average daily traffic (AADT) for the design year
3. The percent of traffic in the peak direction of flow *(D)*
4. The percent of trucks in the traffic stream *(T)*
5. The anticipated ideal free-flow speed of the roadway segment

As the first step, the annual average daily traffic is converted to a directional design hourly volume (DDHV) by using the following equation:

$$\text{DDHV} = \text{AADT} \times K \times D \tag{13-1}$$

The K factor is the percent of AADT that occurs in the peak hour, typically ranging from about 10 percent for urban environments to as much as 20 percent for rural environments. If possible, the K factor used in Eq. 13-1 should be determined by empirical studies performed in the vicinity of the proposed development.

The directional factor D depends on the type of route being planned. Typical values of D are 0.50 for an urban circumferential, 0.55 for an urban radial, and 0.65 for a rural highway. Preferably, the value of D used in Eq. 13-1 should be based on local data.

In the next step of the planning analysis, a per-lane maximum service flow rate SFL_i for a specified level of service i is chosen from Table 13-1. The maximum service flow rate in the table assumes highways with ideal geometric features serving only passenger cars and trucks. The rates are based on an access density of 20 access points per mile, typical of fringe urban and suburban conditions. A peak-hour factor (PHF), defined as the ratio of the total hourly volume to the maximum 15-min rate of flow within the hour, of 0.9 was used.

Finally, N, the number of lanes in each direction, is estimated by the equation

$$N = \frac{\text{DDHV}}{\text{SFL}_i} \tag{13-2}$$

This planning analysis should provide a reasonable preliminary estimate of the number of lanes required based on limited traffic and geometric information normally available in the planning stage. A more refined capacity analysis is normally required during the design phase of the project.

EXAMPLE 13.1 ESTIMATION OF NUMBER OF HIGHWAY LANES

A divided highway is to be built in a suburban environment with rolling terrain. The forecast AADT is 16,000 vehicles/day, with 10 percent trucks. The fraction of traffic in the peak direction of flow is 0.60. The peak-hour factor is 0.90, and the desired level of service is B. Assume that the K factor is 0.15. Determine the number of lanes that will be needed in each direction.

The directional design hourly volume is calculated by multiplying the AADT by the K factor and the directional factor using Eq. 13-1:

$$\text{DDHV} = 16{,}000 \times 0.15 \times 0.60 = 1440 \text{ vehicles/hr}$$

Table 13-1 Service Flow Rates in Vehicles per Lane for Use in Planning Analyses

Type of Terrain	Level of Service	By Percent of Trucks 0%	5%	10%	15%	20%
Level	A	590	580	570	550	540
	B	990	970	940	920	900
	C	1360	1330	1290	1260	1240
	D	1620	1580	1540	1510	1470
	E	1890	1840	1800	1760	1720
Rolling	A	590	540	500	460	420
	B	990	900	830	760	710
	C	1360	1240	1130	1050	970
	D	1620	1470	1350	1250	1160
	E	1890	1720	1580	1450	1350
Mountainous	A	590	480	400	340	300
	B	990	790	660	570	500
	C	1360	1090	910	780	680
	D	1620	1300	1080	930	810
	E	1890	1510	1260	1080	950

Note: Lane widths are 12 ft. Shoulder width is 6 ft. PHF = 0.9. Number of access points = 20 per mile. Divided highway. Free-flow speed = 96 km/hr (60 mph).

Source: Highway Capacity Manual, Special Report 209, Transportation Research Board, Washington, DC, 1994.

From Table 13-1, the maximum service flow rate per lane for level of service (LOS) B is

$$SFL_B = 830$$

By Eq. 13-2, the number of lanes required is

$$N = \frac{1440}{830} = 1.7 \text{ lanes}$$

To maintain a level of service B, two lanes will have to be built in each direction.

13-2. WIDTHS OF HIGHWAY PAVEMENTS

While most design standards permit the lane width to be less than 3.6 m (12 ft), there is general agreement that the 3.6-m (12-ft) width is more desirable. On high-speed and high-volume roads 4.0-m (13-ft) and 4.2-m (14-ft) widths have been used. Widths in excess of 4.2 m (14 ft) are not recommended because some drivers will use the roadway as a multilane facility. On some multilane divided highways combinations 3.6-m (12-ft) and 4.0-m (13-ft) lanes are used, especially where large truck combinations are likely to occur.

In urban areas, recommended pavement width varies primarily with the classification of street or highway and the volume of traffic served. The latter factor depends primarily on the development density. Local residential streets vary in width from about 8.2 to 11.0 m (27 to 36 ft), providing safe movement of one-lane traffic in each direction, even where occasional curb parking can be expected to occur.

A minimum pavement width of 11.0 m (36 ft) is recommended for residential collector streets except in high-density developments where at least 12.2 m (40 ft) of pavement is specified. These widths provide two traffic lanes, one in each direction, plus space for curb parking on each side. By prohibiting curb parking in the vicinity of intersections, an additional lane may be provided to facilitate turning movements.

13-3. DIVIDED HIGHWAYS

To provide protection against the conflict of opposing traffic, highways are frequently divided by a median strip.

The width of these median strips varies from 1.2 to 18.3 m (4 to 60 ft) or more. A median strip less than 1.2 to 1.8 m (4 to 6 ft) in width is considered to be little more than a centerline stripe and its use, except for special conditions, should be discouraged. The narrower the median, the longer must be the opening in the median to give protection to vehicles making left turns at points other than intersections. Where narrow medians must be used, many agencies install median barriers to separate physically opposing streams of traffic to minimize the number of head-on collisions.

A variety of median barriers have been employed successfully, including steel W-beam guard rail, box beam, and steel cable [2]. Concrete barriers with sides specially sloped to redirect errant vehicles are recommended for narrow medians.

While medians of 4.3 to 4.9 m (14 to 16 ft) are sufficient to provide most of the separation advantage of opposing traffic, medians of 4.9 to 18.3 m (16 to 60 ft) are now recommended. The median should also be of sufficient width to maintain vegetation and to support low-growing shrubs that reduce headlight glare of opposing traffic. Median strips at intersections should receive careful consideration and should be designed to permit necessary turning movements.

Divided highways need not be of constant cross section. The median strip may vary in width, the roads may be at different elevations, and the superelevation may be applied separately in each pavement. In rolling terrain, substantial savings may be affected in construction and maintenance costs by this variation in design. This type of design also tends to eliminate the monotony of a constant width and equal grade.

13-4. PARKING LANES AND SIDEWALKS

A parking lane is a lane separate and distinct from the traffic lane. Parking should be prohibited on rural highways, but in some rural areas parking adjacent to the traffic lane cannot be avoided. Parallel parking in this case should be permitted and extra lanes provided for this purpose. Where it is desirable to provide parking facilities in parks, scenic outlooks, or other points of interest, off-the-road parking should be provided. In urban and suburban locations the parking area often includes the gutter section of the roadway and may vary from 1.8 to 2.4 m (6 to 8 ft).

The minimum width of a parking lane for parallel parking is 2.4 m (8 ft), with 3.0 m (10 ft) preferred. For angle parking, the width of the lane increases with the angle. When the angle of parking exceeds 45 degrees, it is necessary to use two moving traffic lanes for maneuvering the vehicle into position. Angle parking should be used only in low-speed urban areas where parking requirements take precedence over the smooth flow of traffic. Parking at the approaches to intersections should be prohibited.

The use of sidewalks in the highway cross section is accepted as an integral part of city streets. In rural areas little consideration has been given to their construction since pedestrian traffic is very light. Serious consideration should be given to the construction of sidewalks in all areas where the number of pedestrians using the highway warrants it. The Institute of Transportation Engineers recommends that sidewalks be provided along subdivision streets where the development density exceeds five dwellings per hectare (two dwellings per acre). Where the residential density warrants the use of sidewalks, a sidewalk width of 1.2 to 1.8 m (4 to 6 ft) is recommended.

13-5. SPECIAL CROSS-SECTIONAL ELEMENTS

In the design of multilane highways and expressways, it is necessary to separate the through traffic from the adjacent service or frontage roads. The width and type of separator is controlled by the width of right-of-way, location and type of overpasses or underpasses, and many other factors. The border strip, which is that portion of the highway between the curbs of the through highway and the frontage or service road, is generally used for the location of utilities.

Special consideration for off-the-road parking facilities, mailbox turnouts, or additional lanes for truck traffic on long grades causes corresponding changes in the cross sections. Highways involving these elements of the cross section usually are treated as special cases and receive consideration at the time of their design.

13-6. RIGHT-OF-WAY

Right-of-way requirements are based on the final design of the cross-sectional elements of the facility. On two-lane secondary highways with an average daily traffic volume of 400 to 1000 vehicles, a minimum of 20 m (66 ft) is required, with 24 m (80 ft) desirable. On the Interstate system minimum widths will vary depending on conditions from 46 m (150 ft) without frontage roads to 76 m (250 ft) with frontage roads. An eight-lane divided highway without frontage roads will require a minimum of 60 m (200 ft), while the same highway with frontage roads will require a minimum of 90 m (300 ft). In rural areas of high-type two-lane highways, a minimum width of 30 m (100 ft) with a desirable width of 36 m (120 ft) is recommended. A minimum of 45 m (150 ft) and a desirable width of 76 m (250 ft) is recommended for divided highways.

A right-of-way width of 18 m (60 ft) is generally recommended for local subdivision streets, while collector streets should have 21 m (70 ft).

Right-of-way should be purchased outright or placed under control by easement or other means. When this is done, sufficient right-of-way is available when needed. This eliminates the expense of purchasing developed property or the removal of other encroachments from the highway right-of-way.

13-7. PAVEMENT CROWN OR SLOPE

Another element of the highway cross section is the pavement crown or slope. This is necessary for the proper drainage of the surface to prevent ponding on the pavement. Pavement crowns have varied greatly through the years. On the early low-type roads, high crowns were about 4.0 percent (½ in. or more per foot). Present-day high-type pavements with good control of drainage now have crowns as low as about 1.0 percent (⅛ in./ft). This has been made possible with the improvement of construction materials, techniques, and equipment that permit closer control. Low crowns are satisfactory when little or no settlement is expected and when the drainage system is of sufficient capacity to remove the water quickly from the traffic lane. When four or more lanes are used, it is desirable to provide a higher rate of crown on the inner lanes in order to expedite the flow of water.

13-8. SHOULDERS

Continuous shoulders should be provided along all highways in order to provide safe operation and to allow full traffic capacity to be developed. Well-maintained, smooth, firm shoulders increase the effective width of the traffic lane as much as 0.6 m (2 ft), as most vehicle operators will drive closer to the edge of the pavement in the presence of adequate shoulder. Shoulders should be wide enough to permit and encourage vehicles to leave the pavement when stopping. The greater the traffic density, the more likely will the shoulder be put to emergency use. A shoulder width of at least 3.0 m (10 ft) and preferably 3.6 m (12 ft) clear of all obstructions is desirable for all heavily traveled and high-speed highways. In mountainous areas where the extra cost of providing wide shoulders may be prohibitive, a minimum width of 1.2 m (4 ft) may be used, but a width of 1.8 to 2.4 m (6 to 8 ft) is preferable. Under these conditions, however, emergency parking strips should be provided at proper intervals. In terrain where guardrails or retaining walls are used, an additional 0.6 m (2 ft) of shoulder should be provided.

The slope of the shoulder should be greater than that of the pavement. A shoulder with a high-type surfacing should have a slope at least 3.0 percent (⅜ in./ft). Sodded shoulders may have a slope as high as 8.0 percent (1 in./ft) in order to carry water away from the pavement.

13-9. ROADSIDES AND SLOPES

Since a large percentage of crashes involve vehicles that run off the roadway, a great deal of care should be given to the design of roadsides and slopes. The roadside area (including the shoulder) should be designed so as to give the errant motorist as much chance as possible to regain control of the vehicle. Highway designers attempt to provide a clear roadside recovery area to prevent or lessen the loss from off-road fixed object or rollover crashes.

Research has shown that a traversable and obstruction-free roadside area of about 9 m (30 ft) should permit recovery of about 85 percent of vehicles out of control. It is understood, however, that, in some circumstances, a 9-m (30-ft) clear zone is insufficient; in other situations, a border that wide is not needed.

The AASHTO [2] recommends clear-zone widths based on traffic volumes and speeds and on roadside slopes. See Table 13-2. These distances provide a range of distances to be considered and not a precise distance to be held as absolute. In selecting an appropriate clear-zone width, the designer should consider economic and environmental factors along with site-specific conditions such as horizontal curvature.

The AASHTO [2] suggests that, when considering clear-zone distances along the outside of horizontal curves, the distances shown in Table 13-2 be multiplied by curve correction factors that increase clear-zone widths as much as 50 percent depending on the design speed and degree of curve. These factors are shown in Table 13-3.

Trees, poles, and other fixed-object hazards should generally be removed from clear-zone areas. Signs and other potential hazards that must be placed near the roadway should provide energy attenuation or be of breakaway design. Where neither removal nor safety modification of a fixed-object hazard is feasible, the installation of a highway guardrail may be warranted.

To allow drivers of errant vehicles to regain control without overturning, designers must pay attention to roadside slopes, both those parallel to the direction of traffic and

Table 13-2 Clear-Zone Distances (m) from Edge of Driving Lane

Design Speed (km/hr)	Design ADT	Fill Slopes 6:1 or Flatter	Fill Slopes 5:1 to 4:1	Fill Slopes 3:1	Cut Slopes 3:1	Cut Slopes 4:1 to 5:1	Cut Slopes 6:1 or Flatter
64 or less	Under 750	2.1–3.0	2.1–3.0	[b]	2.1–3.0	2.1–3.0	2.1–3.0
	750–1500	3.0–3.7	3.7–4.3	[b]	3.0–3.7	3.0–3.7	3.0–3.7
	1500–6000	3.7–4.3	4.3–4.9	[b]	3.7–4.3	3.7–4.3	3.7–4.3
	Over 6000	4.3–4.9	4.9–5.5	[b]	4.3–4.9	4.3–4.9	4.3–4.9
72–80	Under 750	3.0–3.7	3.7–4.3	[b]	2.4–3.0	2.4–3.0	3.0–3.7
	750–1500	3.7–4.3	4.9–6.1	[b]	3.0–3.7	3.7–4.3	4.3–4.9
	1500–6000	4.9–5.5	6.1–7.9	[b]	3.7–4.3	4.3–4.9	4.9–5.5
	Over 6000	5.5–6.1	7.3–8.5	[b]	4.3–4.9	5.5–6.1	6.1–6.7
88	Under 750	3.7–4.3	4.3–5.5	[b]	2.4–3.0	3.0–3.7	3.0–3.7
	750–1500	4.9–5.5	6.1–7.3	[b]	3.0–3.7	4.3–4.9	4.9–5.5
	1500–6000	6.1–6.7	7.3–9.1	[b]	4.3–4.9	4.9–5.5	6.1–6.7
	Over 6000	6.7–7.3	7.9–9.8[a]	[b]	4.9–5.5	6.1–6.7	6.7–7.3
96	Under 750	4.9–5.5	6.1–7.3	[b]	3.0–3.7	3.7–4.3	4.3–4.9
	750–1500	6.1–7.3	7.9–9.8[a]	[b]	3.7–4.3	4.9–5.5	6.1–6.7
	1500–6000	7.9–9.1	9.8–12.2[a]	[b]	4.3–5.5	5.5–6.7	7.3–7.9
	Over 6000	9.1–9.8[a]	11.0–13.4[a]	[b]	6.1–6.7	7.3–7.9	7.9–8.5
104–112	Under 750	5.5–6.1	6.1–7.9	[b]	3.0–3.7	4.3–4.9	4.3–4.9
	750–1500	7.3–7.9	8.5–11.0[a]	[b]	3.7–4.9	5.5–6.1	6.1–6.7
	1500–6000	8.5–9.8[a]	10.4–12.8[a]	[b]	4.9–6.1	6.7–7.3	7.9–8.5
	Over 6000	9.1–10.4[a]	11.6–14.0[a]	[b]	6.7–7.3	7.9–9.1	8.5–9.1

[a]Where a site-specific investigation indicates a high probability of continuing accidents or such occurrences are indicated by accident history, the designer may provide clear-zone distances greater than 9 m as indicated. Clear zones may be limited to 9 m for practicality and to provide a consistent roadway template if previous experience with similar projects or designs indicates satisfactory performance.

[b]Because recovery is less likely on the unshielded, traversable 3:1 slopes, fixed objects should not be present in the vicinity of the toe of these slopes. Recovery of high-speed vehicles that encroach beyond the edge of the shoulder may be expected to occur beyond the toe of the slope. Determination of the width of the recovery area at the toe of the slope should take into consideration right-of-way availability, environmental concerns, economic factors, safety needs, and accident histories. Also, the distance between the edge of the travel lane and the beginning of the 3:1 slope should influence the recovery lane provided at the toe of slope.

Source: From *Roadside Design Guide*. Copyright 1989 by the American Association of State Highway and Transportation Officials. Washington, DC. Used with permission.

Table 13-3 Horizontal Curve Adjustments

Degree of Curve	Curve Correction Factor by *Design Speed*, K_{cz}						
	40 mph	45 mph	50 mph	55 mph	60 mph	65 mph	70 mph
2.0	1.08	1.10	1.12	1.15	1.19	1.22	1.27
2.5	1.10	1.12	1.15	1.19	1.23	1.28	1.33
3.0	1.11	1.15	1.18	1.23	1.28	1.33	1.40
3.5	1.13	1.17	1.22	1.26	1.32	1.39	1.46
4.0	1.15	1.19	1.25	1.30	1.37	1.44	
4.5	1.17	1.22	1.28	1.34	1.41	1.49	
5.0	1.19	1.24	1.31	1.37	1.46		
6.0	1.23	1.29	1.36	1.45	1.54		
7.0	1.26	1.34	1.42	1.52			
8.0	1.30	1.38	1.48				
9.0	1.34	1.43	1.53				
10.0	1.37	1.47					
15.0	1.54						

$CZ_c = (L_c)(K_{cz})$

where CZ_c = clear zone on outside of curvature, m (ft)

L_c = clear-zone distance, m (ft): see Table 13-2.

Clear-zone correction factor is applied to outside of curves only. Curves flatter than 2° do not require an adjusted clear zone.

Source: From *Roadside Design Guide*. Copyright 1989 by the American Association of State Highway and Transportation Officials, Washington, DC. Used with permission.

those at an angle to traffic. Generally speaking, embankment slopes parallel to the direction of traffic flow may be characterized as being recoverable, nonrecoverable, or critical:

1. ***Recoverable slopes*** Embankment slopes 4 (horizontal) to 1 (vertical) are generally considered to be recoverable. This means that motorists encroaching on these slopes can usually stop their vehicle or slow it enough to return safely to the roadway.
2. ***Nonrecoverable slopes*** Slopes between 3:1 and 4:1 are considered to be nonrecoverable. On such slopes, encroaching motorists will be unable to stop or return to the roadway safely. The recovery area on these slopes must extend to the toe of the slope, and a clear runout area with a slope of 6:1 or flatter should be provided at the base.
3. ***Critical slope*** Slopes steeper than 3:1 are considered to be critical slopes. Encroaching vehicles are likely to overturn on these slopes.

The downward slope from the edge of the shoulder toward the ditch is called the front slope. The slope from the edge of the ditch upward is called the back slope. Front slopes of 6:1 or flatter can be negotiated safely by a vehicle and should be provided wherever practical. A 6:1 slope can usually be provided with little extra expense on fills up to about 4.5 m (15 ft) in height. On higher fills, some agencies specify a recovery area having a 4:1 or flatter slope for about 6 m (20 ft), then a steeper slope. This design is known as a barn roof cross section. Much steeper slopes, as steep as 1.5:1, are often used along high-fill sections for economical reasons. In such instances, roadside barriers must be placed along the edge of the fill.

The back slope in cut areas may vary from 1 (vertical) to 6 (horizontal) to vertical in rock to 1.5:1 in normal soil. It is advisable to have back slopes as flat as 4:1 when side borrow is needed. Slope transitions from cut to fill should be gradual and should extend over a considerable length of roadway.

Motorists who leave the roadway may be confronted with embankments or cross slopes at an angle to the flow of traffic. Such slopes are of greater concern to motorists because they are normally struck head-on by out-of-control vehicles. For this reason, milder cross slopes are recommended than for embankments parallel to traffic flow. Along high-speed roadways, such cross slopes should be no steeper than 6:1 and preferably 10:1 or flatter. Embankment cross slopes steeper than 6:1 may be appropriate for low-speed facilities.

13-10. HIGHWAY GUARDRAIL

Generally speaking, guardrail barriers are warranted in fills more than 2.4 m (8 ft) in height with slopes steeper than 3:1. Guardrails may also be required along the edges of deep roadside ditches with steep banks and in areas of limited rights-of-way or steep terrain where steep side slopes must be used. Where guardrails are used, the width of the shoulders should be increased by approximately 0.6 m (2 ft) to allow space for placing the posts.

Various types of guardrail systems are in use at the present time. Three types of guardrails that have performed satisfactorily are shown in Fig. 13-1. References 2 to 5 provide more detailed information on the selection, location, and design of guardrails, median barriers, and energy attenuation devices.

13-11. CURBS, CURB AND GUTTER, AND DRAINAGE DITCHES

The use of curbs generally is confined to urban and suburban roadways. The design of curbs varies from a low, flat, lip-type to a nearly barrier-type curb. Curbs adjacent to traffic lanes, where sidewalks are not used, should be low and very flat. The face of the curb should be no steeper than 45 degrees so that vehicles may mount them without difficulty. Curbs at parking areas and adjacent to sidewalks should be 150 to 200 mm (6 to 8 in.) in height, with faces nearly vertical. Clearances should be sufficient to clear fenders and bumpers and to permit the opening of doors. When a barrier or nonmountable curb is used, it should be offset a minimum of 3 m (10 ft) from the traffic lane.

The side slopes of ditches should be as flat as possible, consistent with drainage requirements. Preferably, the bottoms of ditches should be wide and rounded so as to be traversable by out-of-control vehicles. A rounded ditch section has been found to be safer than a V-type ditch, which also may be subject to severe erosion. Reference 2 has figures that show combinations of back slopes and front slopes of ditches that can be traversed by encroaching vehicles.

13-12. LIMITED-ACCESS HIGHWAYS

A limited-access highway may be defined as a highway or street, designed especially for through traffic, to which motorists and abutting property owners have only a restricted right of access. Limited or controlled-access highways may consist of (1) freeways that are open to all types of traffic and (2) parkways from which all commercial traffic is excluded. Most of our present expressway systems have been developed as freeways.

Limited-access highways may be elevated, depressed, or at grade. Many examples of the various types may be found in the United States in both rural and urban areas.

The control of access is attained by limiting the number of connections to and from the highway, facilitating the flow of traffic by separating cross traffic with overpasses or underpasses and by eliminating or restricting direct access by abutting property owners.

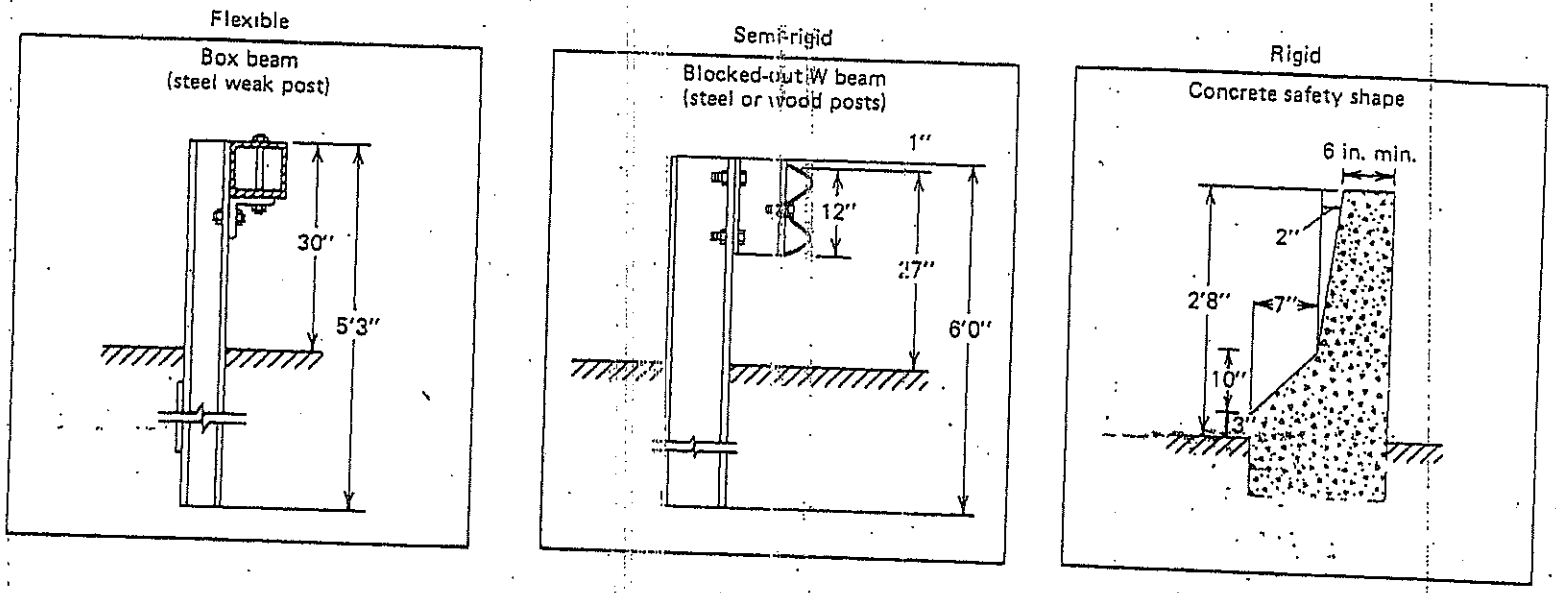


Figure 13-1 Examples of guardrails that have performed satisfactorily. (Source: *Handbook of Highway Safety Design and Operating Practices*, Federal Highway Administration, 1979.)

The design of limited-access routes should provide for adequate width of right-of-way, adequate landscaping, prohibition of outdoor advertising in the controlled-access proper, and provisions for controlling abutting service facilities such as service stations, parking areas, and other roadside appurtenances.

In urban areas, the design of a limited-access facility is often accompanied by the design of frontage roads, parallel to the facility, which may serve local traffic and provide access to adjacent property. Such roads may be designed for either one-way or two-way operation. Connections between through-traffic lanes and frontage roads must be carefully designed to avoid motorist confusion and hazardous traffic conflicts.

13-13. HIGHWAY DESIGN STANDARDS

Design standards vary widely for different functional classes of highways. For example, freeways are designed predominantly for traffic movement, and freeway standards are characterized by high-design speeds, wide lanes, and a gentle change in horizontal and vertical alignments. At the other end of the spectrum, standards for subdivision streets reflect an emphasis on land access function. Such standards, exemplified by Tables 13-4 and 13-5, are based on design speeds of 30 to 55 km/hr (20 to 35 mph). In fact, local streets should be designed to discourage excessive speeds through the use of curvilinear alignment and discontinuities in the street system [4, 6].

Within a functional class, design standards may vary with the type of terrain, anticipated traffic to be served, and whether the highway is to be in an urban or rural area. It is not surprising, therefore, to find some variation in standards among the various highway agencies.

Typical cross-sectional dimensions for arterial streets and rural highways are shown, respectively, in Figures 13-2 and 13-3.

THE RAILWAY CROSS SECTION

The railway cross section of today is the result of more than a century of evolutionary change. In the early days of railroading, both the longitudinal-tie-track and the crosstie-track were used. For the U.S. intercity railroad system, the crosstie-track has prevailed and, predominantly, wood crossties have been used.

The cross-sectional area of the rail has been increased continuously, and the tie spacing has been decreased to accommodate increasing wheel loads. However, the spacing of the ties cannot be reduced indefinitely, and the rail section cannot be increased without limit. For these reasons, concrete ties are replacing wood ties on heavy-traffic main lines, and some thought now is being given to eliminating the tie spaces altogether by using instead a continuous reinforced concrete slab. In this design, the rails would be secured by fasteners that are anchored in the slab [7].

The paragraphs that follow describe the various components of a railroad cross section. Looking to the future it should be remembered that other design concepts and approaches to track design may be needed to serve heavier loads and increased traffic and to adapt to technological change.

Standards for the width of the subgrade are determined for mainline, secondary-line, and light-traffic branch lines and spurs. Many railroads have adopted 6.1 m (20 ft) as the width of single-track mainlines. This width will also vary according to the height of fill. A width of 6.7 m (22 ft) has been used for fills under 6.1 m (20 ft), 7.3 m (24 ft) for fills 6.1 to 15.2 m (20 to 50 ft), and 7.9 m (26 ft) for fills over 15.2 m (50 ft). The standard width in cuts including the side ditches is 9.1 m (30 ft). In some locations 15.2 m (40 ft) is used

Table 13-4 Collector Street Design Guidelines by Low, Medium, and High Development Intensity

Terrain classification	Level Terrain			Rolling Terrain			Hilly Terrain		
Development density	Low	Medium	High	Low	Medium	High	Low	Medium	High
Right-of-way width (ft)[a]	70	70	70	70	70	70	70	70	70
Pavement width (ft)	36	36	40	36	36	40	36	36	40
Type of curb (V = vertical face)	V	V	V	V	V	V	V	V	V
Sidewalk width (ft)	4–6	4–6	4–6	4–6	4–6	4–6	4–6	4–6	4–6
Sidewalk distance from curb face (ft)	10	10	10	10	10	10	10	10	10
Minimum sight distance (ft)	250	250	250	200	200	200	150	150	150
Maximum grade (%)	4	4	4	8	8	8	12	12	12
Minimum spacing along major traffic route (ft)	1300	1300	1300	1300	1300	1300	1300	1300	1300
Design speed (mph)[b]	35	35	35	30	30	30	25	25	25
Minimum centerline radius (ft)[c]	350	350	350	250	250	250	175	175	175
Minimum tangent between reverse curves (ft)	100	100	100	100	100	100	100	100	100

[a] 1 ft = 0.3048 m.

[b] 1 mph = 1.6093 km/hr.

[c] Assumes superelevation.

Source: *Recommended Guidelines for Subdivision Streets: A Recommended Practice*, Institute of Transportation Engineers, Washington, DC, © 1984. Used by permission.

Table 13-5 Local Street Design Guidelines

Terrain classification	Level			Rolling			Hilly		
Development density	Low	Medium	High	Low	Medium	High	Low	Medium	High
Right-of-way width (ft)[a]	50	60	60	50	60	60	50	60	60
Pavement width (ft)	22–27	28–34	36	22–27	28–34	36	28	28–34	36
Type of curb (V = vertical face, R = roll-type, 0 = none)	0/R	V	V	V	V	V	V	V	V
Sidewalks and bicycle paths (ft)	0	4–6	4–6	0	4–6	4–6	0	4–6	4–6
Sidewalk distance from curb face (ft)	—	6	6	—	6	6	—	6	6
Minimum sight distance (ft)	200	200	200	150	150	150	110	110	110
Maximum grade (%)	4	4	4	8	8	8	15	15	15
Maximum cul-de-sac length (ft)	1000	700	700	1000	700	700	1000	700	700
Minimum cul-de-sac radius (right-of-way) (ft)	50	50	50	50	50	50	50	50	50
Design speed (mph)[b]	30	30	30	25	25	25	20	20	20
Minimum centerline radius of curves (ft)	250	250	250	175	175	175	110	110	110
Minimum tangent between reverse curves (ft)	50	50	50	50	50	50	50	50	50

[a] 1 ft = 0.3048 m.

[b] 1 mph = 1.6093 km/hr.

Source: *Recommended Guidelines for Subdivision Streets: A Recommended Practice*, Institute of Transportation Engineers, Washington, DC, © 1984. Used by permission.

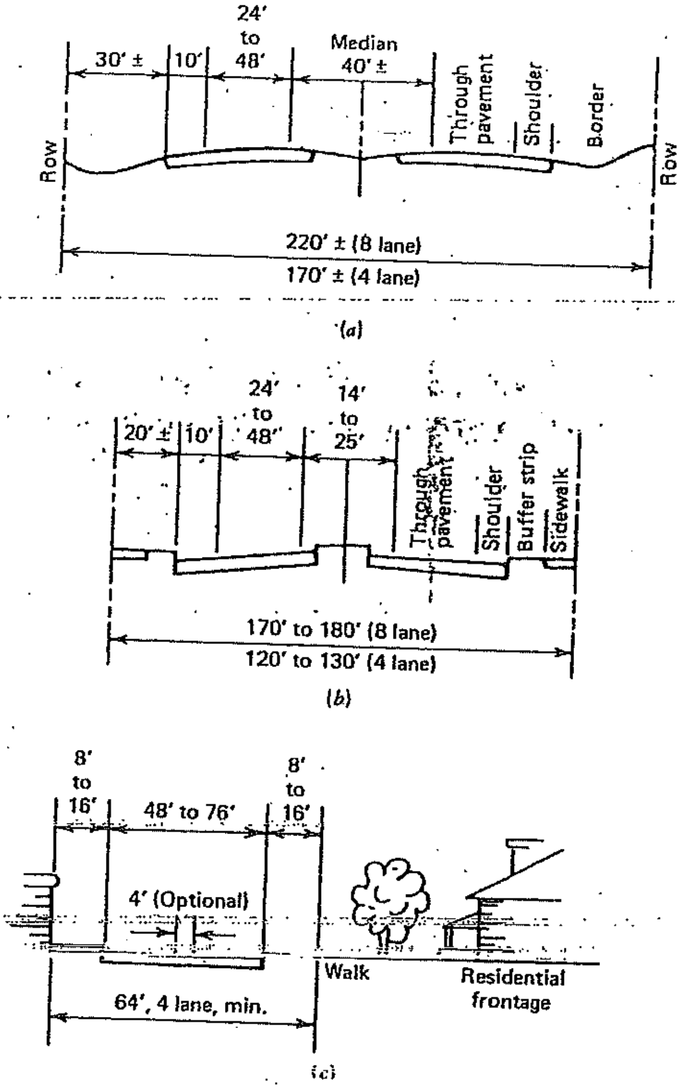


Figure 13-2 Typical cross-section dimensions for arterial streets. (*a*) Desirable. (*b*) Intermediate. (*c*) Restricted without frontage roads. (Source: *A Policy on Geometric Design of Highways and Streets*, copyright 1994, American Association of State Highway and Transportation Officials, Washington, DC. Used by permission.)

to permit the use of off-track equipment for maintenance purposes. Common widths of rights-of-way in open country are 15.2, 18.3, 24.3, and 30.5 m (50, 60, 80, and 100 ft). Occasionally, wider rights-of-way are used.

There are seven main elements of a railroad cross section: (1) ballast, (2) crossties, (3) rails, (4) tie plates, (5) fastenings, (6) rail anchors, and (7) rail joints.

13-14. BALLAST

Track ballast is a key structural element of the railroad permanent way. Its prime function is to transmit and distribute the wheel loadings from the base of the crossties to the subgrade at pressures that will not cause subgrade failure. In addition, ballast serves to anchor the track, preventing longitudinal and transverse track movements under dynamic train loading, to provide immediate drainage of the permanent way under the ties, and to provide a road material that inhibits vegetation growth and minimizes dust.

TYPICAL CROSS-SECTIONS

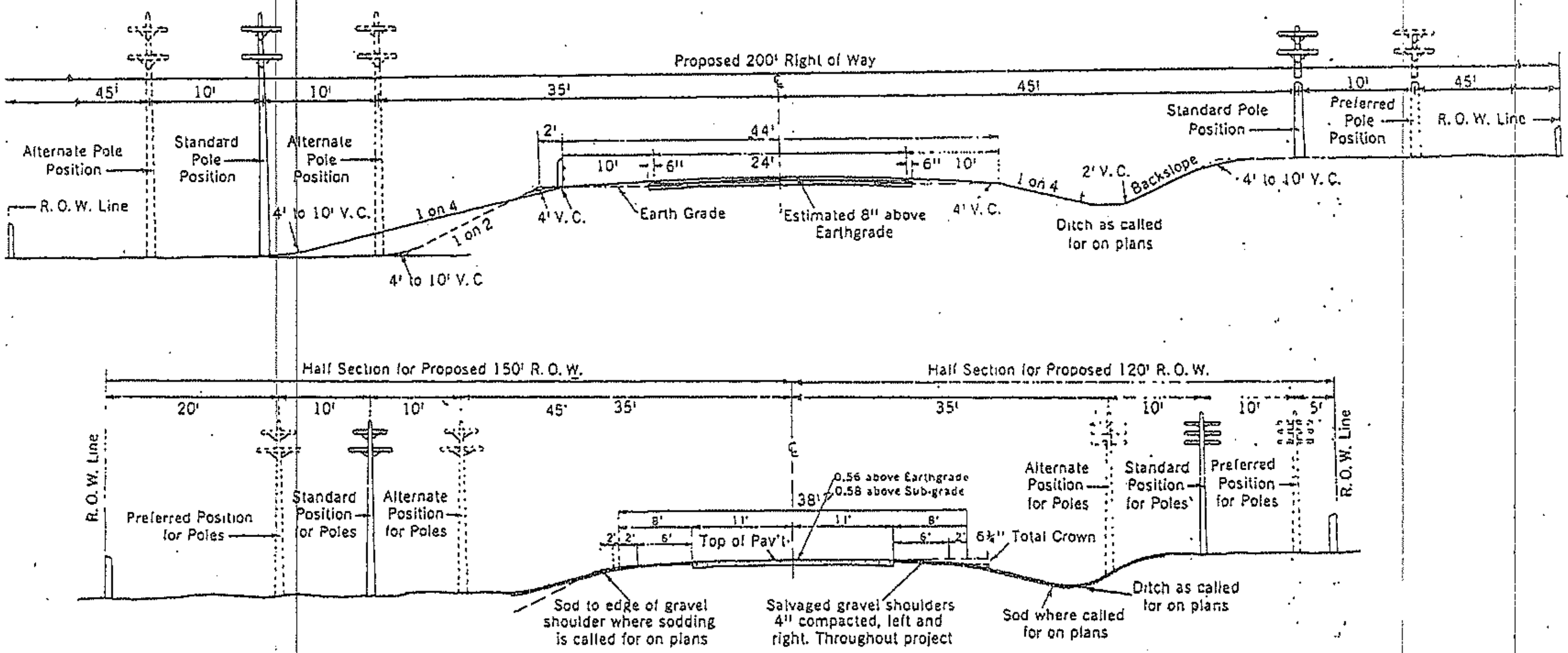


Figure 13-3 Typical highway cross sections. (Courtesy Michigan Department of State Highways and Transportation.)

Open graded materials that can perform satisfactorily the required functions of ballast are crushed stone, washed river or pit run gravel, and furnace slags. Typically, material varies in grain size from 38 to 44 mm (1½ to 1¾ in.). Where ballast material is expensive or in short supply or where subgrade strength is sufficiently low that excessive depths of ballast would be required, a layer of subballast frequently is used. Material for the subballast layer of the permanent way can be a less openly graded material meeting less stringent quality requirements.

The depth of the ballast may vary from 150 to 750 mm (6 to 30 in.) or more depending on wheel loads, traffic density and speed, and the type and condition of the foundation. The thickness of the subballast may also vary, but excellent results have been obtained with a thickness of about 300 mm (12 in.) [8].

The AREA has set quality standards on ballast with reference to the following criteria:

1. ***Wear resistance*** Under the Los Angeles abrasion test, percentage of wear of any ballast material is limited normally to 40 percent.
2. ***Cleanliness*** Deleterious substances are limited in prepared ballasts to the following amounts: anchor rail / drainage water / inhibit vegetation

   a. Soft and friable pieces, 5 percent
   b. Material finer than No. 200 sieve, 1 percent
   c. Clay lumps, 0.5 percent

3. ***Frost resistance*** Ballast must be capable of resisting freeze–thaw cycles. The AREA requires an average weight loss of not more than 7 percent after five cycles of the sodium sulfate soundness test.
4. ***Unit weight*** Specifications require compacted weights of not less than 1120 to 1600 kg/m³ (70 and 100 lb/ft³) for blast furnace and open-hearth slags, respectively.

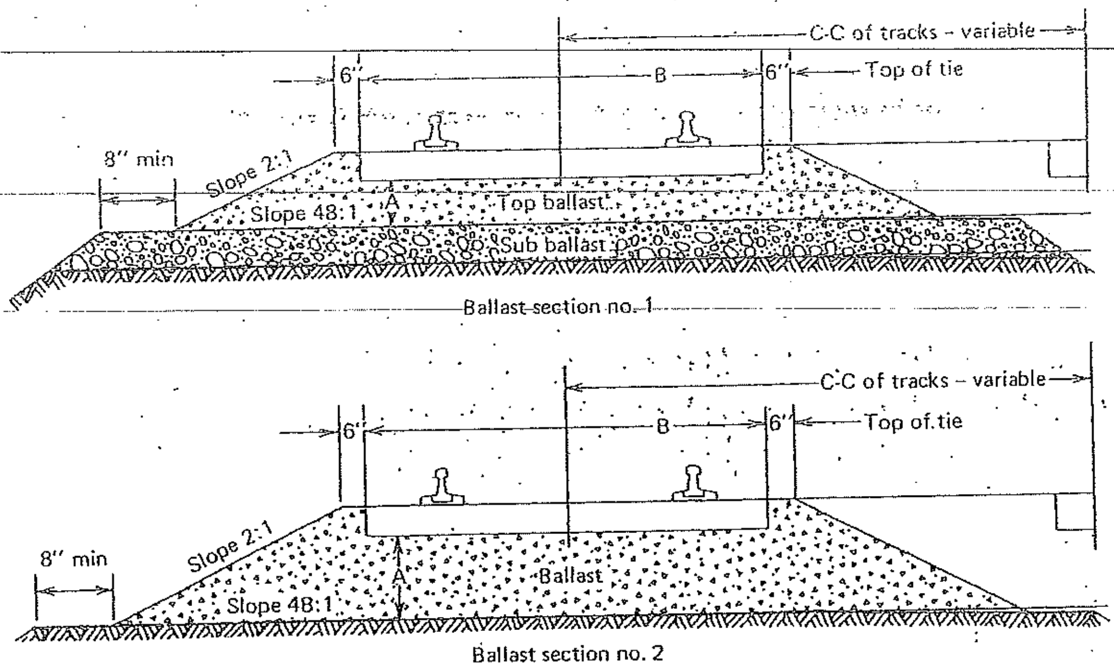
13-15. CROSSTIES (SLEEPERS)

The crosstie serves several functions, including (1) spreading the horizontal and vertical loadings to the ballast; (2) maintaining the correct gauge between the rails; (3) providing, in conjunction with the ballast, a means of anchoring the track against longitudinal and lateral movements; (4) deflecting under load to minimize the impact of loads on rails and equipment; and (5) providing a convenient means for making needed adjustments of the vertical profile of the track [8].

Vertical loadings are applied to the ties by the train weight. Horizontal longitudinal loadings occur as trains accelerate and decelerate, while transverse loadings are applied as the vehicles traverse curved sections. Additional transverse loads are present due to the "barreling" effect of locomotives at high speed. To permit the horizontal transfer of forces from the tie, the ballast is tamped mechanically between the ties, as shown in Fig. 13-4.

Typically, ties are made of wood that is treated with both preservative and coating materials for protection against weathering and splitting while in service. U.S. railroads use two standard cross sections for wood ties: 7-in. grade (nominal 7 in. thick × 9 in. wide) and 6-in. grade (nominal 6 in. thick × 8 in. wide). Standard lengths are 2.4, 2.6, and 2.7 m (8, 8.5, and 9 ft). Tie replacement, which averages about 3 percent of all ties yearly, accounts for a large proportion of total track maintenance.

Many railroads have had satisfactory experience with concrete crossties (*"sleepers"* in British terminology). Reference 9 reports that there are more than 3000 million ties in the world, of which more than 400 million are concrete. Concrete ties are desirable because they do not decay, and the heavier mass helps to keep them in place under heavy traffic.



Area ballast sections, single and multiple track, tangent

Notes:
Depth of ballast section to be used will depend on conditions peculiar to each railroad or location.
Sections apply to all types of ballast.
Sections for use with jointed or continuous rail.
Top of ballast determined by the use of various mechanized ballast distributing operations.

Figure 13-4 Typical roadbed section for single main track. (Courtesy Southern Railway System.)

Elastic pads under the rails provide the resilience needed for rail traffic. In fact, concrete ties account for more than half of the crosstie installation demand in many parts of the world, especially in most of Europe, in Russia, in Japan, and in some parts of Africa [9].

Two types of design systems have been used for concrete ties to accommodate the wide range of positive and negative bending moments. In the first type, there is one rigid concrete block under each rail connected by a central flexible (e.g., steel) piece. See Fig. 13-5. In the other type, known as the "monoblock" system, a single rigid concrete beam is used. The monoblock ties are almost always prestressed to resist the dynamic bending

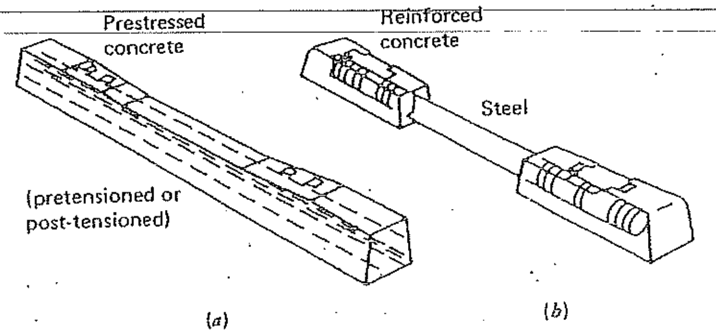


Figure 13-5 Basic types of concrete sleepers. (Source: *Concrete Railway Sleepers, State of the Art Report*, Thomas Telford, London, 1987.)

moment distribution [9]. Concrete sleepers also must be renewed, typically at a rate of 2 to 5 percent per year.

Statistics of the Association of American Railroads indicate that there are approximately 795 million crossties in service in the United States, an average of about 3020 ties/mile of track [10]. This gives an average spacing of approximately 530 mm (21 in.). However, most railroads space ties more closely on mainline tracks and more widely on branch lines and in yards.

13-16. RAILS

The steel rail has a characteristic inverted T-shape. It functions as a continuous steel beam, transmitting vertical loads and horizontal shears to the ties via the tieplates and fastenings. Rail sections come in standard lengths of 11.9 m (39 ft), a length chosen because it can be transported conveniently in a single 12-m (40-ft) car. In recent years, continuous welded rail in lengths of 439 m (1440 ft) has been used increasingly by a number of railroads. Advantages claimed for the use of continuous welded rail include decreased maintenance of way costs, higher permissible operating speeds, less likelihood of damage to lading, and a smoother ride resulting in less wear and tear on equipment.

Rails are designated in standard sections by weight in pounds per yard (e.g., 115 lb/yd). The AREA [8] recommends six rail sections ranging from 115 to 140 lb/yd. There is no established formula for selecting the size of rail. Railroad officials generally base the selection of a rail section on the annual gross tons of traffic to be transported and on the train speeds over the track. A typical rail section is shown in Fig. 13-6.

Track gauge, the distance between the inside rail heads, is a standard 4 ft 8½ in. (1435 mm) in the United States and many other countries. Gauge standardization permits free interchange of rolling stock on a nationwide and international basis.

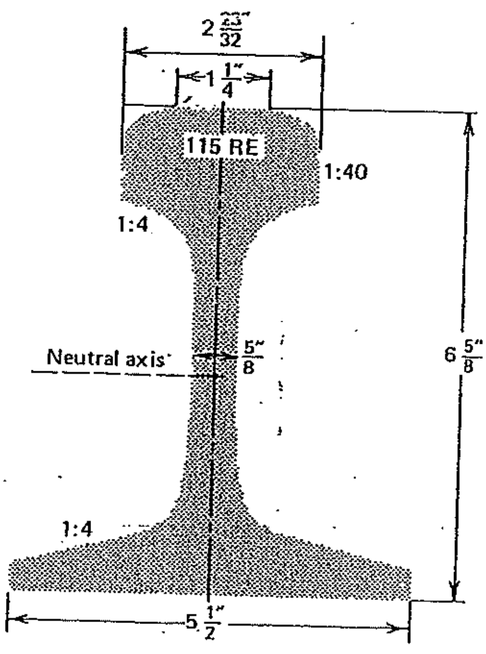


Figure 13-6 Typical rail section. (Courtesy Southern Railway System.)

13-17. TIE PLATES

The rail is laid on tie plates that are secured to the crossties by spikes or other fastenings. A tie plate may be seen in Fig. 13-10 in Section 13-20.

Tie plates have three principal functions [10]:

1. To prevent damage to the wood crosstie by distributing the wheel loads over a larger area
2. To help hold the rail to proper gauge
3. To partially offset the outward lateral thrust of the wheel loads by tilting the rails slightly inward

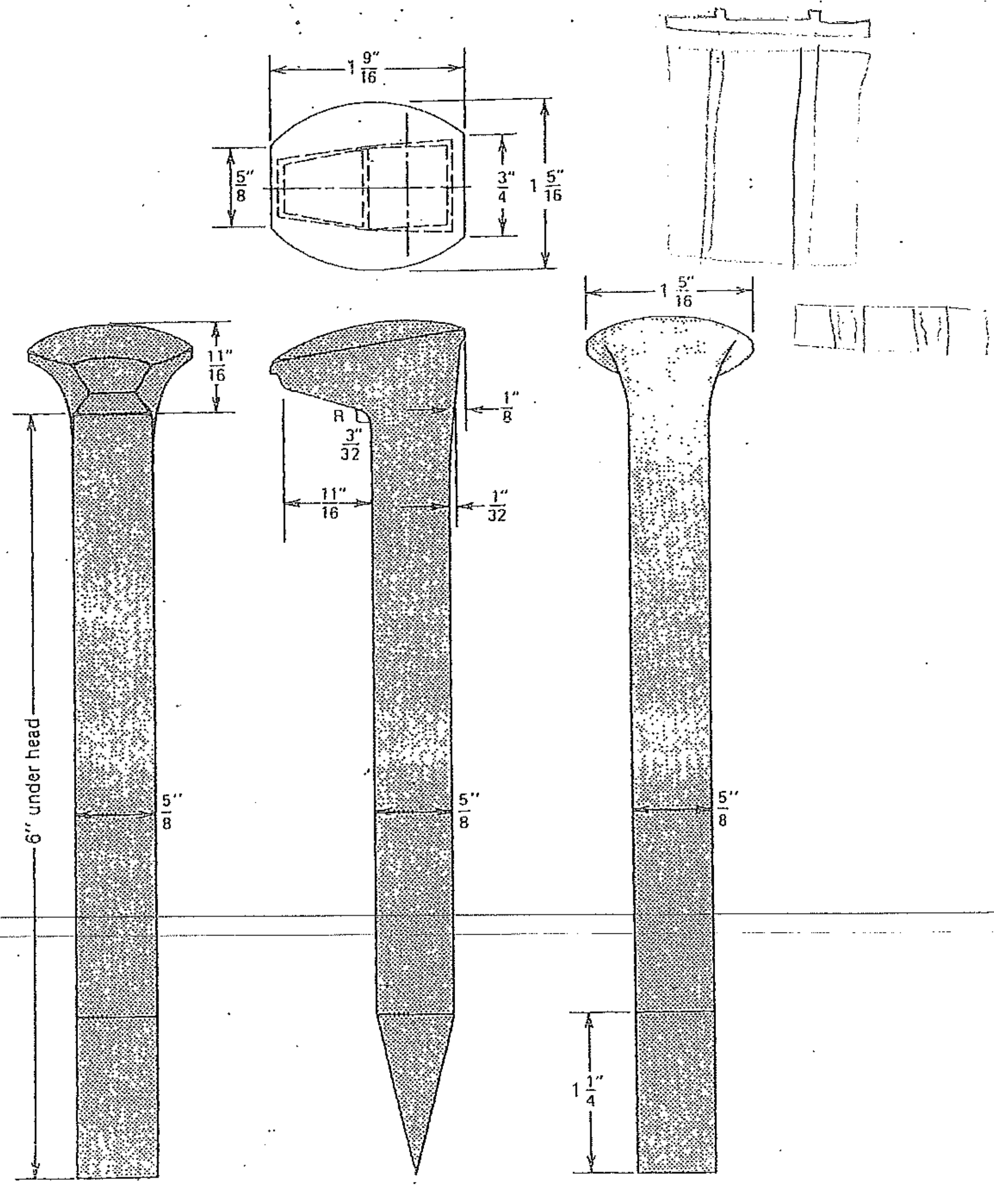


Figure 13-7 Design of a ⅝-in. reinforced throat track spike. (Source: *Manual For Railway Engineering*, American Railway Engineering Association, 1995.)

Tie plates are typically 178 to 203 mm (7 to 8 in.) wide, 254 to 356 mm (10 to 14 in.) long, and 14 to 25 mm (9⁄16 to 1 in.) thick.

Special rubber, neoprene, or plastic pads sometimes are placed between the tie plates and crossties to provide softer riding track and reduce tie wear.

13-18. FASTENINGS

Tie plates must be anchored to the tie to prevent destructive abrasion from plate movement. Several types of fastenings may be used to secure the tie plates, including common cut spikes (Fig. 13-7), spring spikes, screw spikes, compression clips (Fig. 13-8), and clamps.

ادوات ربط بين plate و Cross ties

13-19. RAIL ANCHORS

Rail anchors, exemplified by those shown in Fig. 13-9, are employed to reduce or stop the longitudinal movement of rails under traffic and to control temperature-induced expansion of rails. Their main purpose is to hold the rail in a fixed position with respect to the tie. Without anchorage, rails tend to expand unevenly, and local concentrations of expansion forces may cause the track to buckle or warp. This can result in twisted ties, tight gauge, and broken welds at rail joints. Rail anchors are indispensible when welded rail is used.

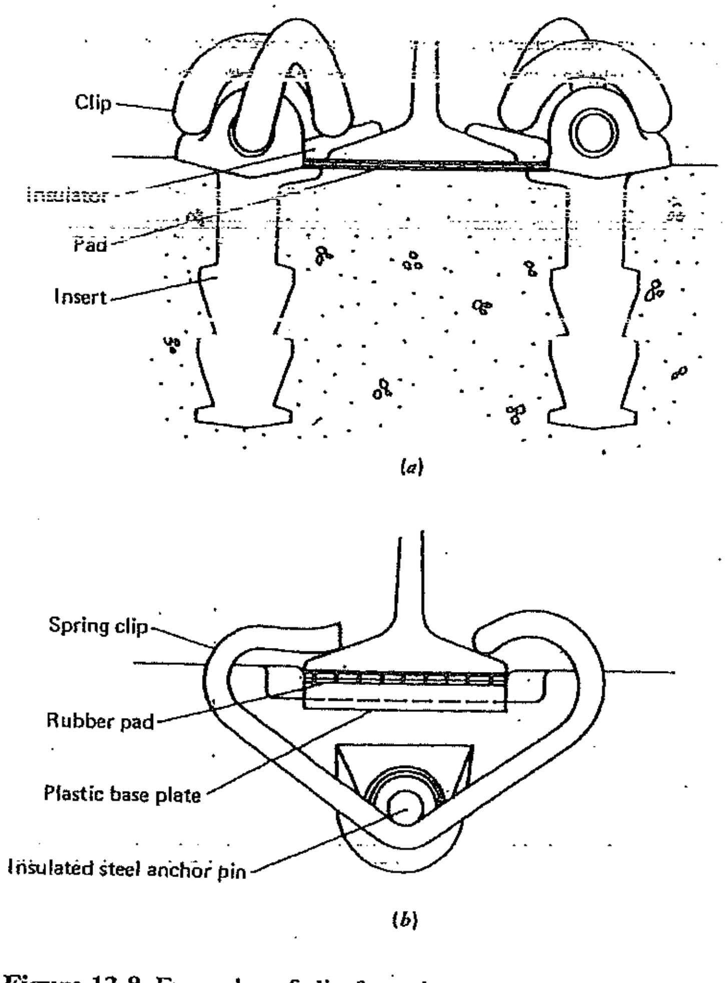
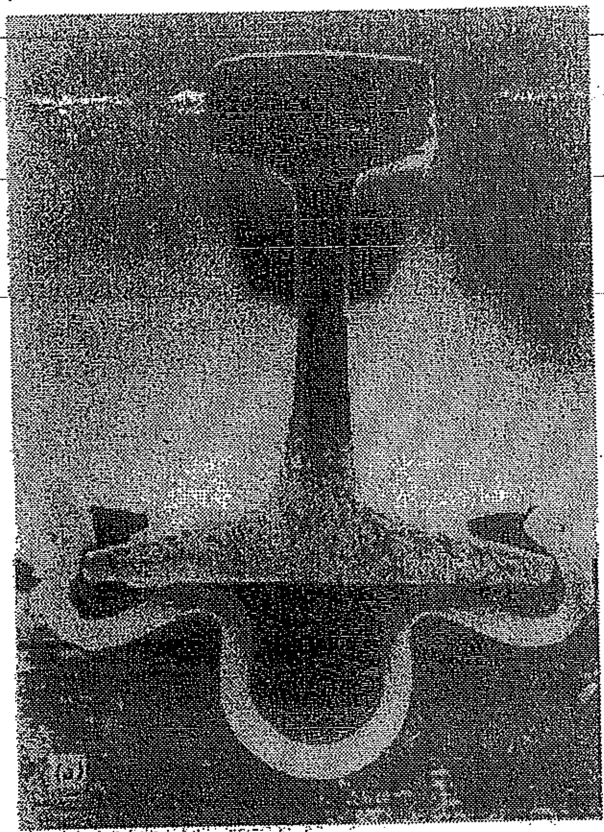


Figure 13-8 Examples of clip fastening systems. (*a*) Pandrol. (*b*) Fist. (Source: *Concrete Railway Sleepers, State of the Art Report,* Thomas Telford, London, 1987.)



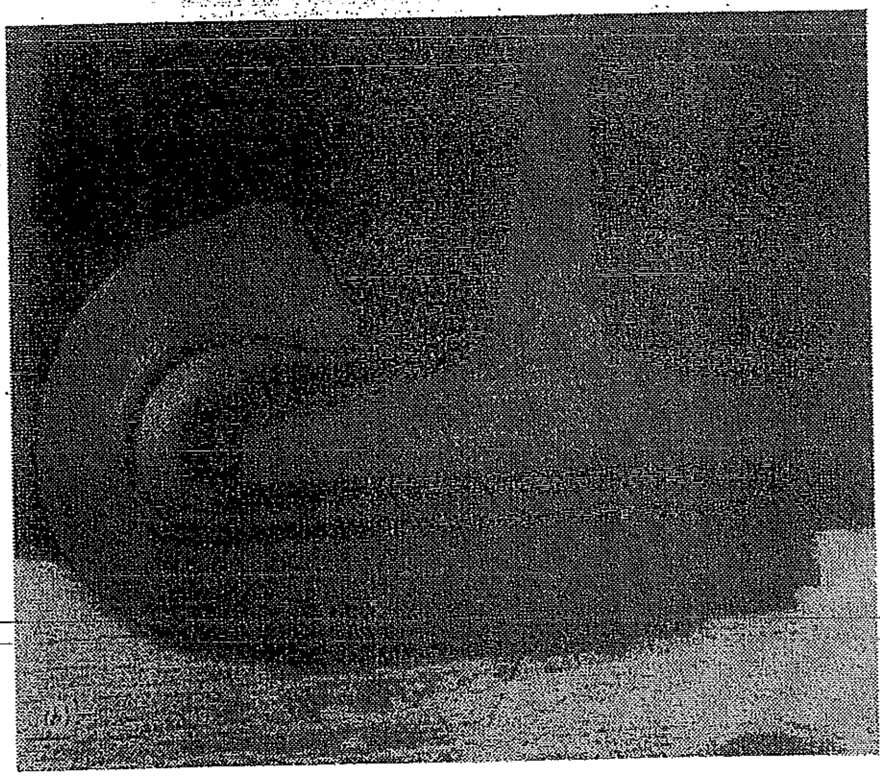


Figure 13-9 Types of rail anchors. (Photos courtesy of (*a*) Woodings-Verona, (*b*) Portec, Inc.)

The rail anchor usually is attached to the base of the rail with one of its vertical surfaces bearing against the side of the tie or tie plate or both. Creeping pressures of the rail are transmitted to the tie and ultimately to the ballast.

Rail anchors are used in great numbers. For mainline tracks carrying traffic essentially in one direction, the AREA recommends that 8 forward anchors and 2 backup anchors be used per 11.9 m (39 ft) of rail length. For mainline tracks carrying traffic in both directions, 16 anchors per 11.9 m (39 ft) of rail length are recommended. Where continuous welded rail is used, each rail usually is anchored at alternate ties [10].

13-20. RAIL JOINTS

Rail joints are used to provide smooth continuity of alignment and surface where two rail ends meet and to transfer the wheel load from one rail end to the other. Rail joints, illustrated by Fig. 13-10, consist of two steel members that fit on each side of the rail and span the gap between the two rails. These bars are typically 610 or 914 mm (24 or 36 in.) in length and usually are held in place by bolting through holes in the rail flange. The 610-mm (24-in) bars have four bolt holes, and the 914-mm (36-in.) bars have six.

Where it is necessary to electrically isolate the track circuit carried by the rails, insulated joints are used. (See Chapter 5 for a discussion of automatic block signaling.) This type of joint has an insulating material placed between the bars and the rail. Some railroads now use a glued or bonded insulated joint, in which the joint bars are fastened rigidly to the rail by means of structural adhesives.

URBAN RAIL TRANSIT CROSS SECTION

The cross sections of urban rail transit systems are essentially similar to conventional railroad systems where duorail systems are used. However, with lighter cars and less demanding service requirements, urban systems may employ prestressed concrete ties with specially designed tie plates, fastenings, anchors, and joints. Atlanta's rail system [the Metropolitan Atlanta Rapid Transit Authority (MARTA)], for example, has 115-lb rail attached to prestressed concrete ties by Pandrol rail clips (illustrated in Fig. 13-8). In the stations, each rail is attached to 660-mm- (2-ft, 2-in.-) wide concrete slabs by means of Hixon direct fixation fasteners. The concrete slabs are typically 3 m (10 ft) in length, and

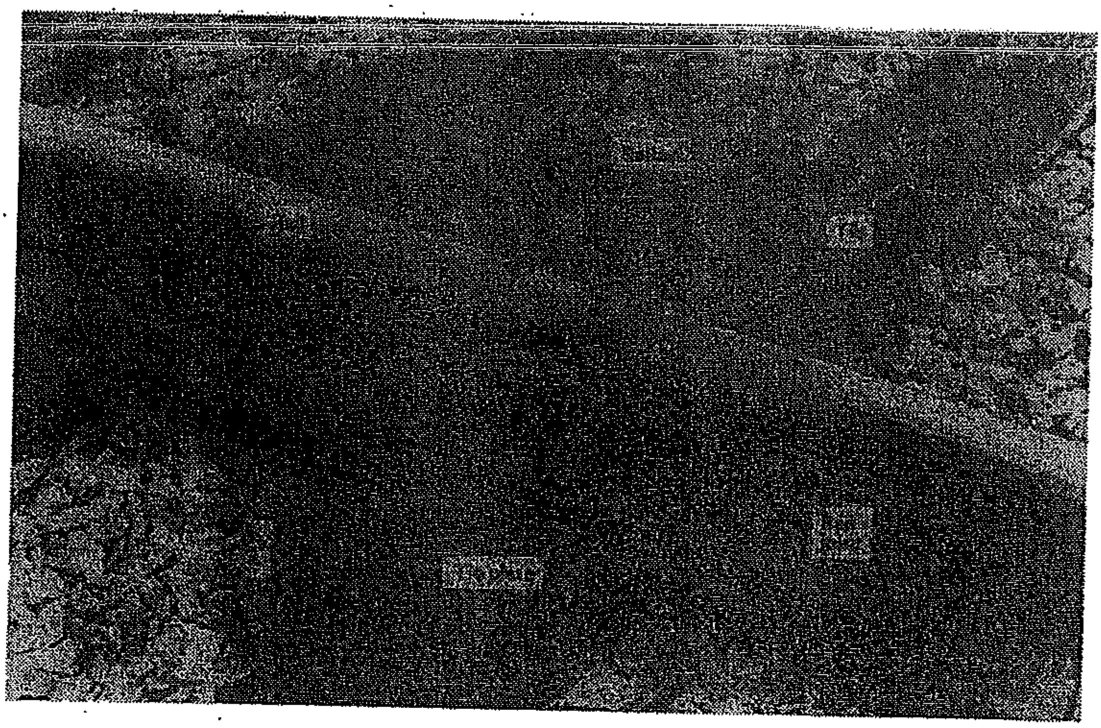


Figure 13-10 A rail joint. (Courtesy, Portec, Inc.)

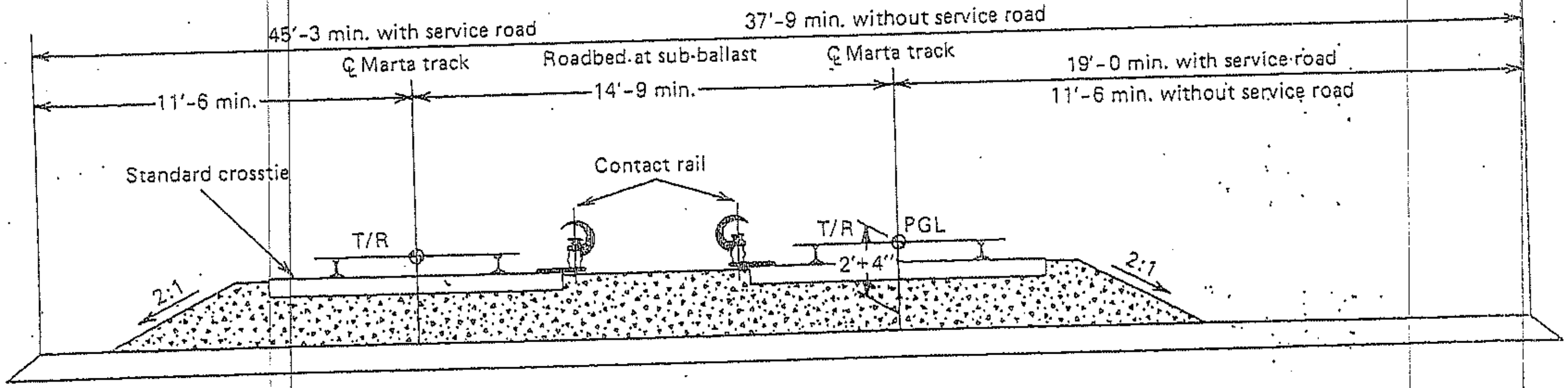


Figure 13-11 A typical section of an urban rail transit track, at grade. (Courtesy Metropolitan Atlanta Rapid Transit Authority.)

the fasteners are spaced in 762 mm (30-in.) centers. A typical section of the MARTA primary track at grade is shown in Fig. 13-11, while typical station sections are shown in Fig. 13-12.

Special cross sections are necessary for nonconventional designs such as suspended and rubber-tired systems. For example, Montreal's transit system utilizes rubber-tired cars that travel along a 254-mm- (10-in.-) wide slab while being guided by small horizontally mounted rubber-tired wheels. A cross section of track for the Montreal system is shown in Fig. 13-13.

In designing the alignment of rail transit and limited-access freeways, the engineer faces the choice of elevated, at-grade, depressed, and subway sections.

At-grade alignments, being the cheapest to construct, have large economic advantages. This type of facility causes the most community disruption from the viewpoint of severing existing street patterns. Noise and fume levels can also be high.

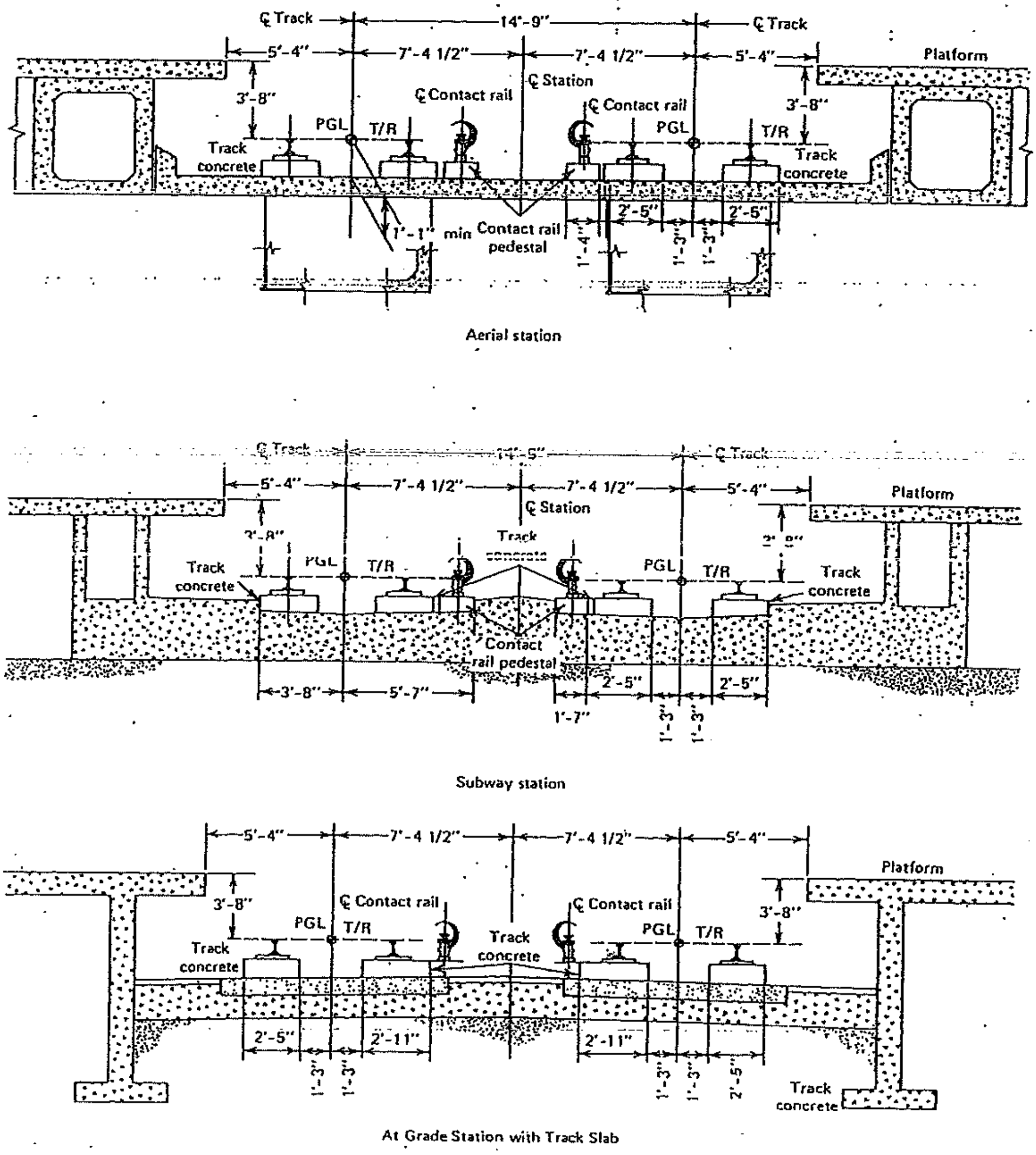


Figure 13-12 Typical sections of urban rail tracks in stations. (Courtesy Metropolitan Atlanta Rapid Transit Authority.)

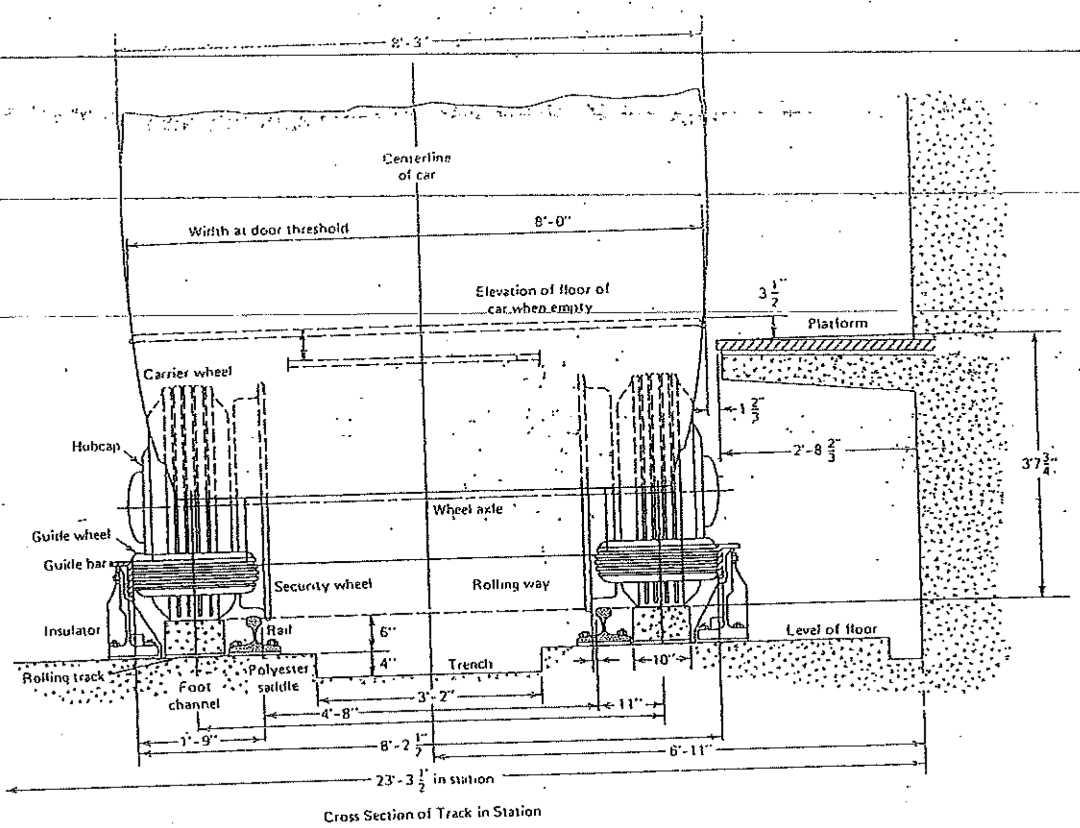


Figure 13-13 Cross section of track in station, for Montreal's rubber-tired transit system.

Depressed systems tend to cut noise and fume levels and, depending on the number of bridge structures provided, can decrease the disruption of existing traffic patterns. Depressed facilities are more expensive to construct than at-grade alignments.

Elevated facilities can eliminate almost completely surface traffic disruption. However, noise, privacy loss, and aesthetic pollution levels tend to be high in spite of the designer's best efforts to make the structures acceptable. In high-density areas these facilities become uneconomic due to the difficulty of providing station space for the rail system at major street intersections.

Subway alignments cause minimum disruption of existing circulation. Their chief disadvantages lie in the very large cost of construction and the increased noise levels within the vehicles.

~~HIGHWAY INTERSECTIONS AND INTERCHANGE DESIGN~~

An important part of a highway is the intersection. This is the place where two or more highways meet and provides an area for the cross movement of vehicle traffic. The efficiency, safety, speed, cost of operation, and capacity are dependent upon its design.

There are three general types of intersections: (1) intersections at grade, (2) grade separations without ramps, and (3) interchanges.

13-21. HIGHWAY INTERSECTIONS AT GRADE

Most highways intersect at grade, and the intersection area should be designed to provide adequately for turning and crossing movements, with appropriate consideration given to alignment, grades, sight distance, and traffic control.

Simple intersections at grade consist of three, four, or more road approaches. A junction of three approaches forms a "branch," T, or Y. A branch is a minor roadway that intersects a main highway at a small deflection angle. A T-intersection is one in which two roads intersect to form a continuous highway and the third road intersects at or nearly at right angles. A Y-intersection is one in which three roads intersect at nearly equal angles. In addition to these types, a flared intersection may be used, which has additional traffic lanes at the intersection area.

The design of the edge of pavement for a simple intersection should provide sufficient clearance between the vehicle and the other traffic lanes. It is frequently assumed that all turning movements at intersections are accomplished at speeds of less than 15 km/hr (10 mph), and the design is based on the physical characteristics of the assumed vehicle. For 90-degree turns, a minimum curb radius of 9 m (30 ft) for passenger car traffic is recommended, and a radius of at least 15 m (50 ft) is required for single-unit trucks. Larger vehicles are best served by three-centered compound curves. Tables 13-6 and 13-7 give recommended minimum edge of pavement designs for turns at intersections using simple curves and three-centered compound curves, respectively.

Table 13-6 Minimum Edge of Pavement Designs for Turns at Intersections: Simple Curves and Tapers

Angle of turn (deg)	Design Vehicle	Simple Curve Radius (m)	Simple Curve Radius with Taper: Radius (m)	Offset (m)	Taper (m:m)
75	P	11	8	0.6	10:1
	SU	17	14	0.6	10:1
	WB-12	—	18	0.6	15:1
	WB-15	—	20	1.0	15:1
	WB-19	—	43	1.2	20:1
	WB-20	—	43	1.3	20:1
	WB-29	—	26	1.0	15:1
	WB-35	—	42	1.7	20:1
90	P	9	6	0.8	10:1
	SU	15	12	0.6	10:1
	WB-12	—	14	1.2	10:1
	WB-15	—	18	1.2	15:1
	WB-19	—	36	1.2	30:1
	WB-20	—	37	1.3	30:1
	WB-29	—	25	0.8	15:1
	WB-35	—	35	0.9	15:1
105	P	—	6	0.8	8:1
	SU	—	11	1.0	10:1
	WB-12	—	12	1.2	10:1
	WB-15	—	17	1.2	15:1
	WB-19	—	35	1.0	30:1
	WB-20	—	35	1.0	30:1
	WB-29	—	22	1.0	15:1
	WB-35	—	28	2.8	15:1

Source: From *A Policy on Geometric Design of Highways and Streets*. Copyright 1994 by the American Association of State Highway and Transportation Officials. Washington, DC. Used with permission.

Table 13-7 Minimum Edge-of-Pavement Designs for Turns at Intersections: Three-Centered Compound Curves

Angle of turn (deg)	Design Vehicle	3-Centered Compound Symmetric Curve Radii (m)	3-Centered Compound Symmetric Offset (m)	3-Centered Compound Asymmetric Curve Radii (m)	3-Centered Compound Asymmetric Offset (m)
75	P	30–8–30	0.6	—	—
	SU	36–14-36	0.6	—	—
	WB-12	36–14–36	1.5	36–14–60	0.6–2.0
	WB-15	45–15–45	2.0	45–15–69	0.6–3.0
	WB-19	134–23–134	4.5	43–30–165	1.5–3.6
	WB-20	128–23–128	3.0	61–24–183	0.3–3.0
	WB-29	76–24–76	1.4	30–24–91	0.5–1.5
	WB-35	213–38–213	2.0	46–34–168	0.5–3.5
90	P	30–6–30	0.8	—	—
	SU	36–12–36	0.6	—	—
	WB-12	36–12-36	1.5	36–12–60	0.6–2.0
	WB-15	55–18–55	2.0	36–12–60	0.6–3.0
	WB-19	120–21–120	3.0	48–21–110	2.0–3.0
	WB-20	134–20–134	3.0	61–21–183	0.3–3.4
	WB-29	76–21–76	1.4	61–21–91	0.3–1.5
	WB-35	213–34–213	2.0	30–29–168	0.6–3.5
105	P	30–6–30	0.8	—	—
	SU	30–11–30	1.0	—	—
	WB-12	30–11–30	1.5	30–17–60	0.6–2.5
	WB-15	55–14–55	2.5	45–12–64	0.6–3.0
	WB-19	160–15–160	4.5	110–23–180	1.2–3.2
	WB-20	152–15–152	4.0	61–20–183	0.3–3.4
	WB-29	76–18–76	1.5	30–18–91	0.5–1.8
	WB-35	213–29–213	2.4	46–24–152	0.9–4.6

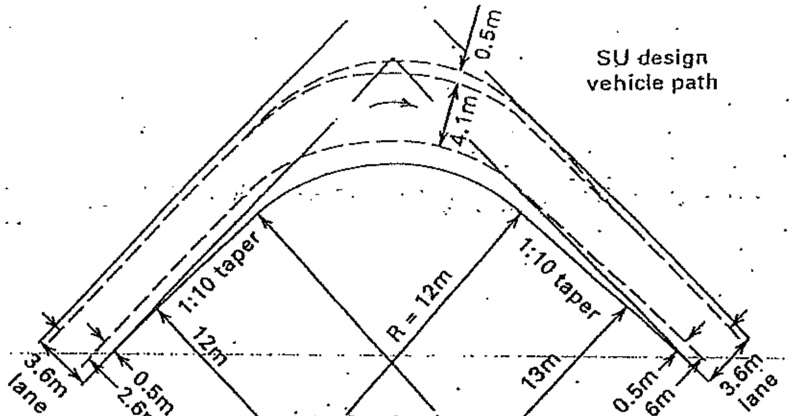
Source: From *A Policy on Geometric Design of Highways and Streets*. Copyright 1994 by the American Association of State Highway and Transportation Officials. Washington, DC. Used with permission.

Figure 13-14 shows examples of minimum designs for the inner edge of a traveled way for at-grade intersections. The figure shows (a) a simple curve with tapers, (b) a symmetric three-centered compound curve, and (c) an asymmetric three-centered compound curve.

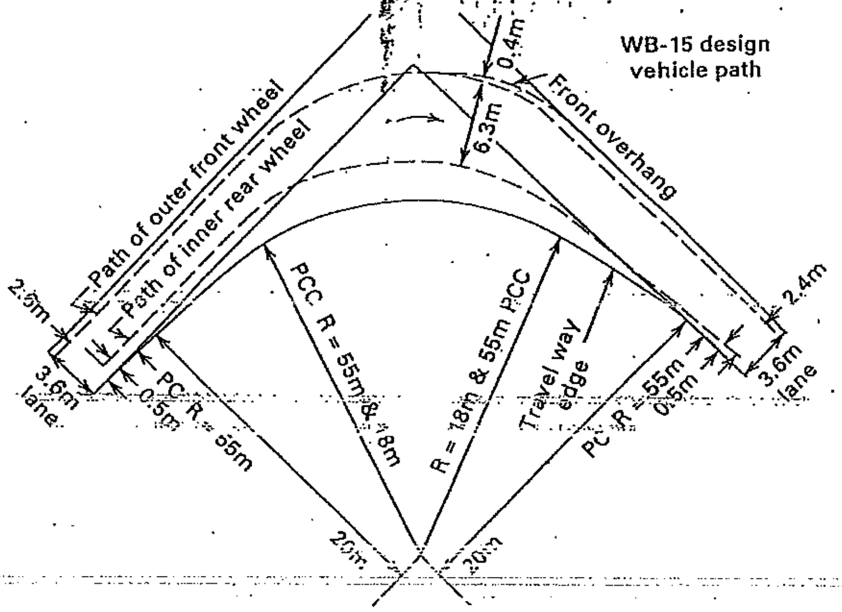
13-22. CHANNELIZED INTERSECTIONS

Channelization is defined as the separation of conflicting traffic movements into definite paths of travel by means of markings, raised islands, or other suitable means to facilitate the safe and orderly movements of both vehicles and pedestrians.

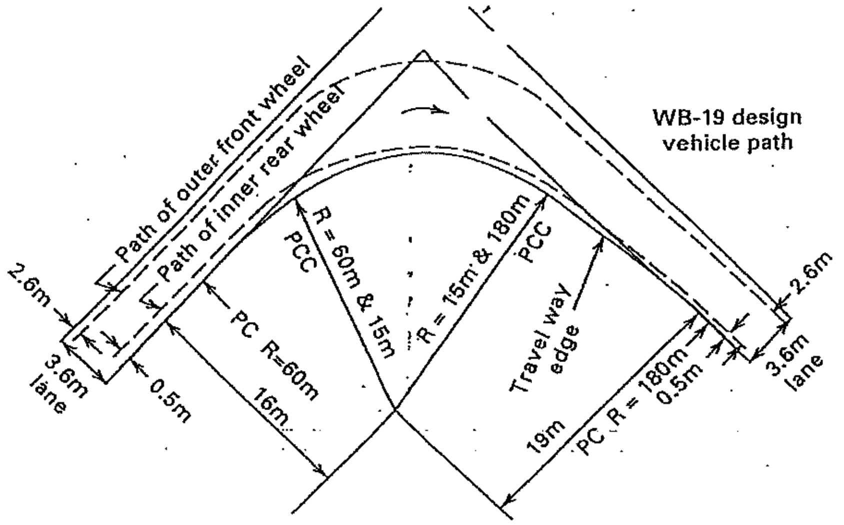
Islands in an intersection can separate conflicting movements or control the angle at which conflict may occur. They can regulate traffic, indicate the proper use of the intersection, and often reduce the amount of pavement required. They provide for the protection of motorists, protection and storage of turning and crossing vehicles, and space for traffic control devices.



Minimum simple curve
with taper
12m radius, offset 1m

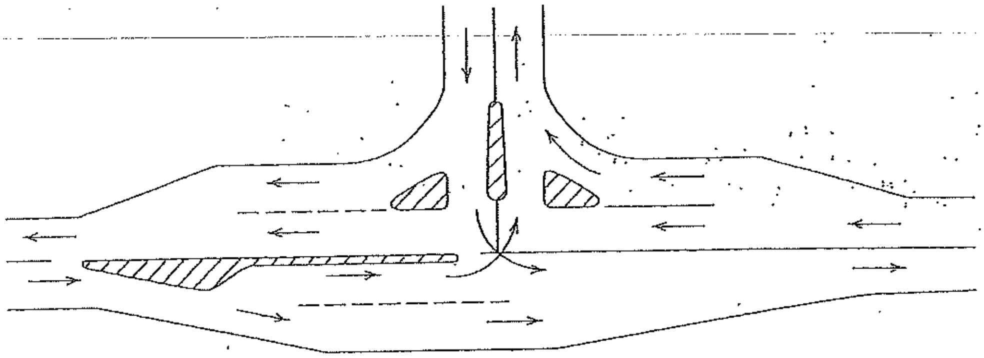


WB-15 semitrailer combination
3 centered compound curve,
55m–18m 55m Radii offset 2m



WB-19 interstate semitrailer combination
3-centered compound curve
60m–15m–180m Radii
offset 1m and 4m

~~Figure 13-14 Examples of minimum designs for inner-edge of travelled way. (*a*) Simple curve with tapers. (*b*)~~ Symmetrical 3-centered compound curve. (*c*) Assymetrical 3-centered compound curve. (Source: *A Policy on Geometric Design of Highways and Streets*, copyright 1994, American Association of State Highway and Transportation Officials, Washington, DC. Used by permission.)



With divisional island and turning roadways

Figure 13-15 Channelized "T" intersection showing general types and shapes of islands. (Source: *A Policy on Geometric Design of Highways and Streets*, copyright 1994, American Association of State Highway and Transportation Officials, Washington, DC. Used by permission.)

Islands are generally grouped into three major classes: directional, divisional, and refuge. General types and shapes of islands are shown in Fig. 13-15.

Directional islands are designed primarily to guide the motorists through the intersection by indicating the intended route. The placing of directional islands should be such that the proper course of travel is immediately evident and easy to follow. A complicated system of islands where the desired course of travel is not immediately evident may result in confusion and may be more hindrance than help in maintaining a steady traffic flow. Islands should be so placed that crossing streams of traffic will pass at approximately right angles and merging streams of traffic will converge at flat angles. By the use of such angles there will be less hindrance to traffic in the thoroughfare and possibility of accidents in the intersection will be decreased.

Divisional islands are used most frequently in individual highways approaching intersections. They serve to alert the driver to the intersection and regulate the flow of traffic into and out of the intersection. Their use is particularly advantageous for controlling left-~~turning traffic at skewed intersections.~~

A refuge island is located at or near crosswalks to aid and protect the pedestrian. These islands are used most generally on wide streets in urban areas for loading and unloading transit riders. The design of refuge islands is the same as that of other types of islands, except that a higher barrier curb is necessary.

A good approach in the design of channelization is to make a comprehensive study of field conditions. Then, with the use of pavement markings, observation of the traffic patterns is done. This is followed by the placing of sand bags and another observation of the traffic pattern, which is then followed by the placing of the permanent channelization.

13-23. ROTARY INTERSECTIONS

One approach to the channelization of traffic is by the use of a rotary intersection. A rotary intersection is one in which all traffic merges into and emerges from a one-way road around a central island. Rotary intersections provide continuous traffic movement but at relatively low speeds. However, experience has shown that a rotary can handle no more traffic than a well-designed channelized intersection. Furthermore, rotaries require relatively large expanses of flat land and are not suitable where large amounts of pedestrian traffic exist. Nevertheless, with proper design and proper traffic control devices in place, the rotary does have numerous applications. In fact, several state DOTs have reported recent and successful installations of rotary intersections.

13-24. GRADE SEPARATIONS AND INTERCHANGES

Intersections at grade can be eliminated by the use of grade separation structures that permit the cross flow of traffic at different levels without interruption. The advantage of such separation is the freedom from cross interference with resultant savings of time and increase in safety for traffic movements.

Grade separations and interchanges may be warranted (1) as a part of an express highway system designed to carry heavy volumes of traffic, (2) to eliminate bottlenecks, (3) to prevent accidents, (4) where the topography is such that other types of design are not feasible, and (5) where the volumes to be catered to would require the design of an intersection, at grade, of unreasonable size.

An interchange is a grade separation in which vehicles moving in one direction of flow may transfer direction by the use of connecting roadways. These connecting roadways at interchanges are called ramps.

Many types and forms of interchanges and ramp layouts are used in the United States. These general forms may be classified into four main types:

1. T- and Y-interchanges
2. Diamond interchanges
3. Partial and full cloverleafs
4. Directional interchanges

T- and Y-Interchanges. Figure 13-16 shows typical layouts of interchanges at various junctions. The geometry of the interchange can be altered in favor of certain movements by the provision of large turning radii and to suit the topography of the site. The trumpet interchange has been found suitable for orthogonal or skewed intersections. Figure 13-16*a* favors the left turn on the freeway by the provision of a semidirect connecting ramp. Figure 13-16*c* indicates an intersection where all turning movements are facilitated in this way. The intersection shown in Fig. 13-16*b* is an example of a poor design. In the upper (north) approach, the southbound driver is confronted with a confusing choice of exits while facing traffic from the exit ramp from the east. At point x, there is an unconventional left-handed entrance ramp followed by an extremely short weaving section where eastbound traffic exiting to the north must merge with southbound traffic exiting to the east.

Diamond Interchanges. The diamond interchange is adaptable to both urban and rural use. The major flow is grade separated, with turning movements to and from the minor flow achieved by diverging the merging movements with through traffic on the minor flow. Only the minor-flow directions have intersection at grade. In rural areas this is generally acceptable, owing to the light traffic on the minor flow. In urban areas, the at-grade intersections generally will require signalized control to prevent serious interference of ramp traffic and the cross arterial street. The design of the intersection should be such that the signalization required does not impair the capacity of the arterial street. To achieve this, widening of the arterial may be necessary in the area of the interchange. Care must

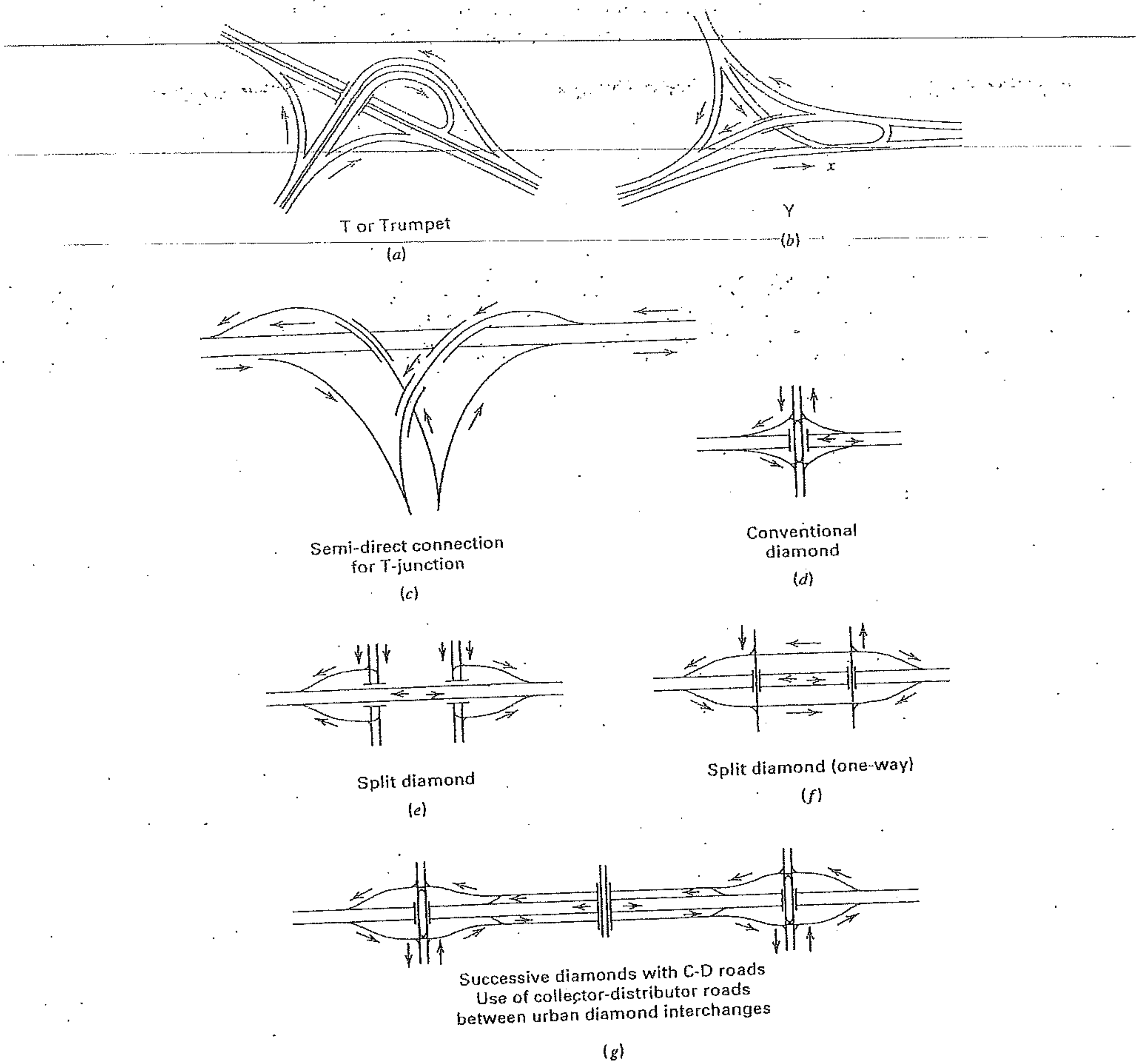


Figure 13-16 Highway interchanges. (Partially adapted from "Adaptability of Interchanges to Interstate Highways," *Transactions*, American Society of Civil Engineers, Vol. 124, pg 558.)

also be taken in the design of the ramps, so that traffic waiting to leave the ramp will not back up into through lanes of the major flow.

One disadvantage of the diamond interchange is the possibility of illegal wrong-way turns, which can cause severe accidents. Where the geometry of the intersection may lead to these turns, the designer can use channelization devices and additional signing and pavement marking. Wrong-way movements are, in general, precluded by the use of cloverleaf designs.

Figure 13-16*d* shows the conventional diamond interchange. Increased capacity of the minor flows can be attained by means of the arrangement shown in Fig. 13-16*e* or Fig. 13-16*f*. The arrangement shown in Fig. 13-16*g* is suitable where two diamond ramps are in proximity. Weaving movements that in this case would inhibit the flows of the major route are transferred to the parallel collector–distributor roads. Figure 13-17 shows a typical depressed expressway in an urban area.

Partial and Full Cloverleafs. The partial cloverleaf shown in Fig. 13-18 is sometimes adopted in place of the diamond interchange. Traffic can leave the major flow either before or after the grade separation structure, depending on the quadrant layout. The intersections at grade for the minor road are present as for the diamond interchange, but the probability of illegal turning movements can be reduced. By the provision of two on-ramps for each direction of the major route as in Fig. 13-18*c*, left-turn traffic on the minor route can be eliminated. The more conventional arrangement of the full cloverleaf, which

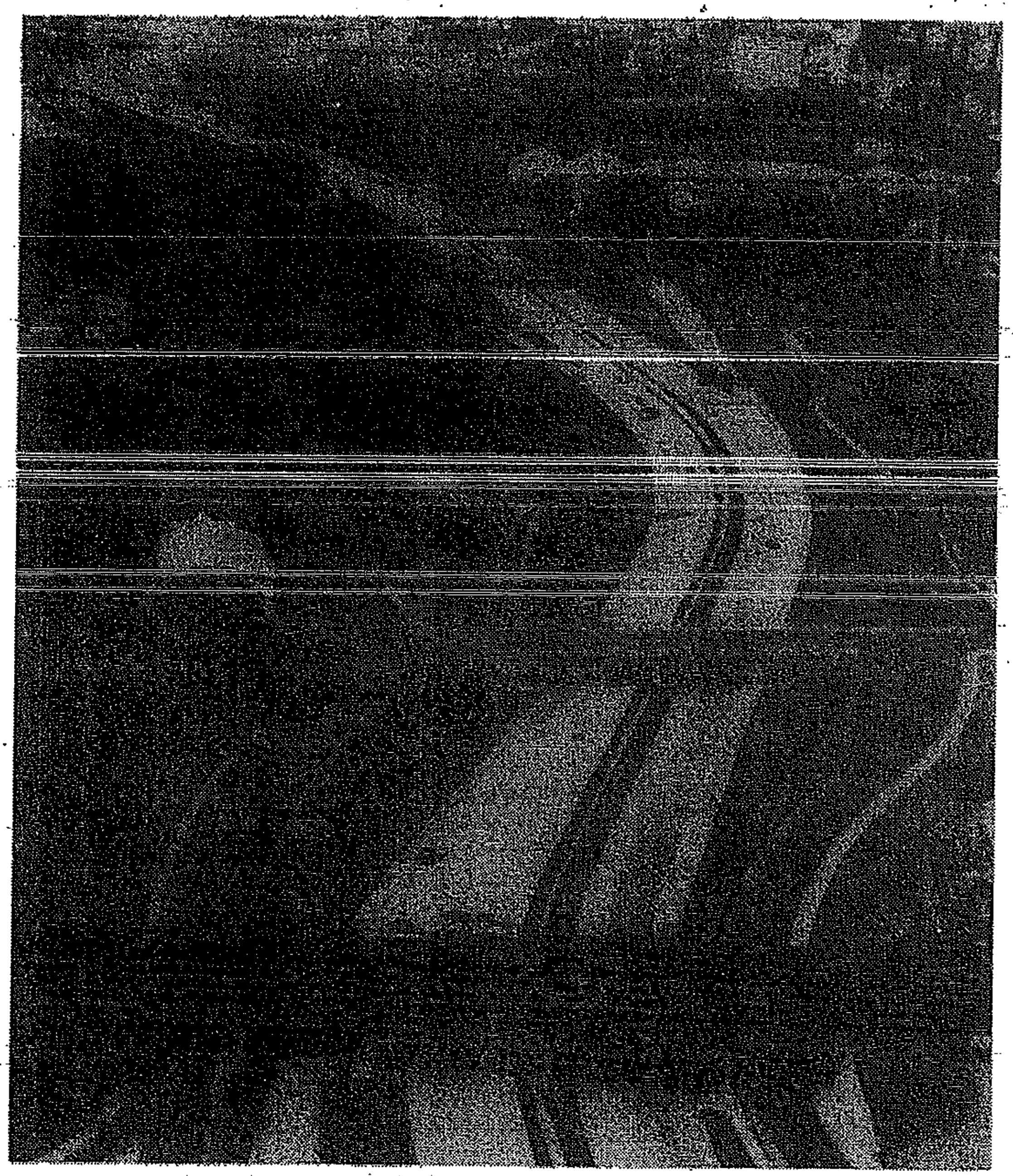


Figure 13-17 A typical depressed expressway in an urban area. (Courtesy Federal Highway Administration.)

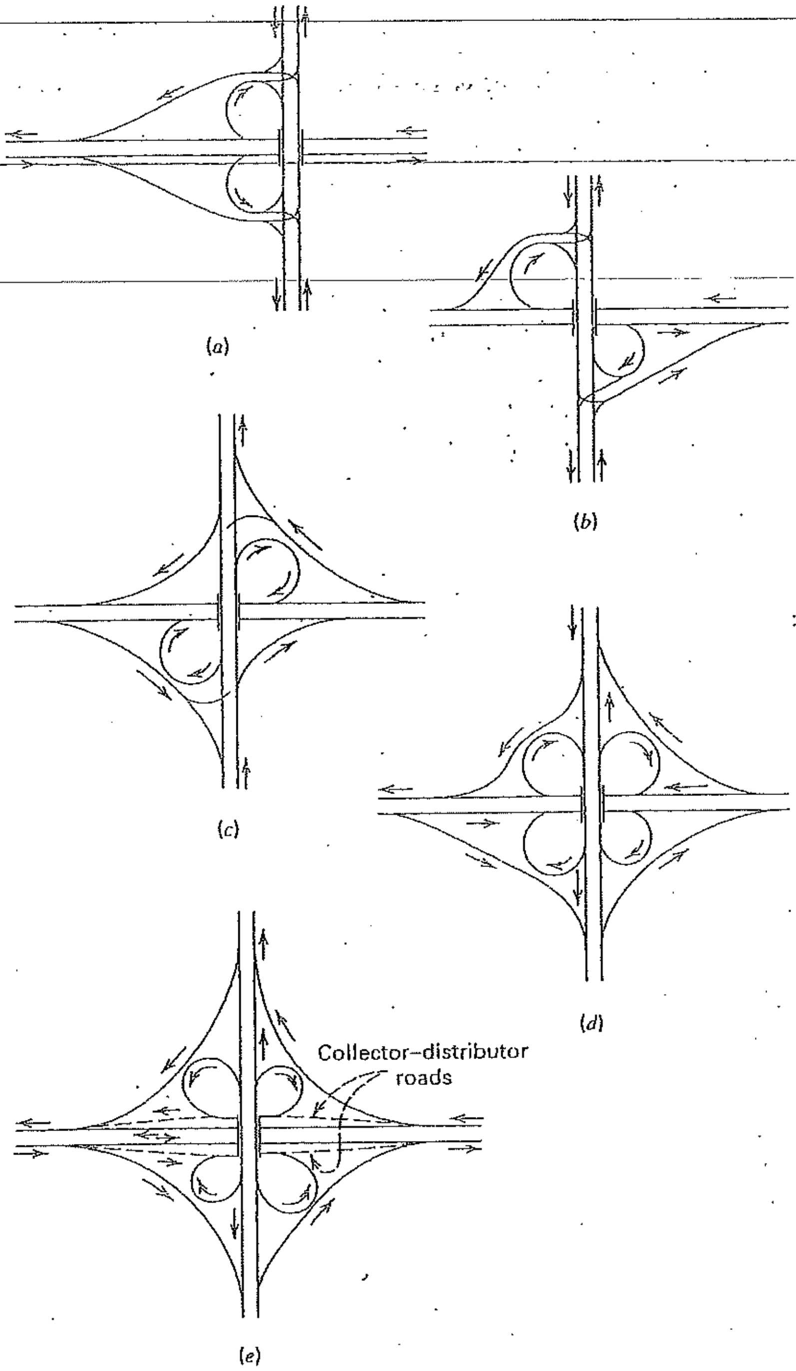


Figure 13-18 Cloverleaf interchanges. (Source: *A Policy on Geometric Design of Rural Highways*, copyright 1965, American Association of State Highway and Transportation Officials, Washington, DC. Used by permission.).

can be adapted to nonorthogonal layouts, eliminates at-grade crossings of all traffic streams for both major and minor roads. The ramps may be one-way, two-way separated, or two-way unseparated roads. Although all crossing movements are eliminated, the cloverleaf design has some disadvantages: (1) the layout requires large land areas and (2) decelerating traffic wishing to leave the through lanes must weave with accelerating traffic entering the through lanes. Figure 13-18*e* is a layout using collector–distributor roads to overcome this second disadvantage.

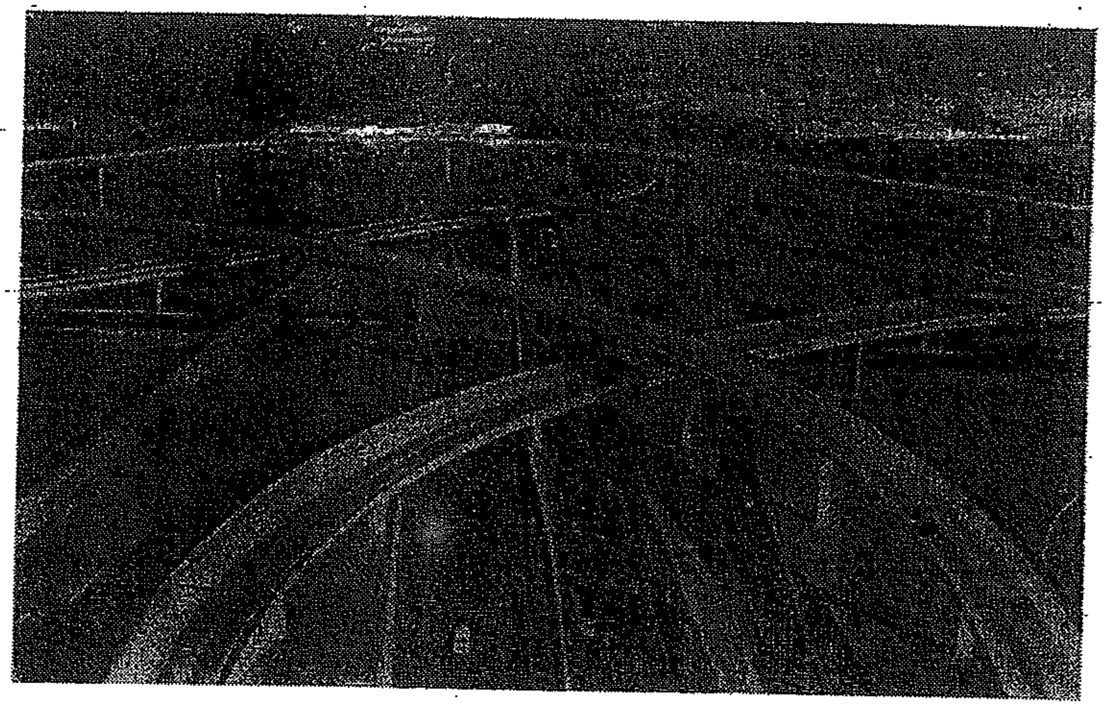


Figure 13-19 The Tom Moreland interchange, Atlanta, Georgia. (Photo by Nick Arroyo.)

Directional Interchanges. Directional interchanges are used whenever one freeway joins or intersects another freeway. The outstanding design characteristic of this type of interchange is the use of a high-design speed throughout, with curved ramps and roadways of large radius. The land requirements for a directional interchange, therefore, are very large. In cases where volumes for certain turning movements are small, design speeds for these movements are reduced and the turnoff is effected within a loop. In the highest type of design weaving sections are eliminated. Figure 13-19 shows a multilevel interchange.

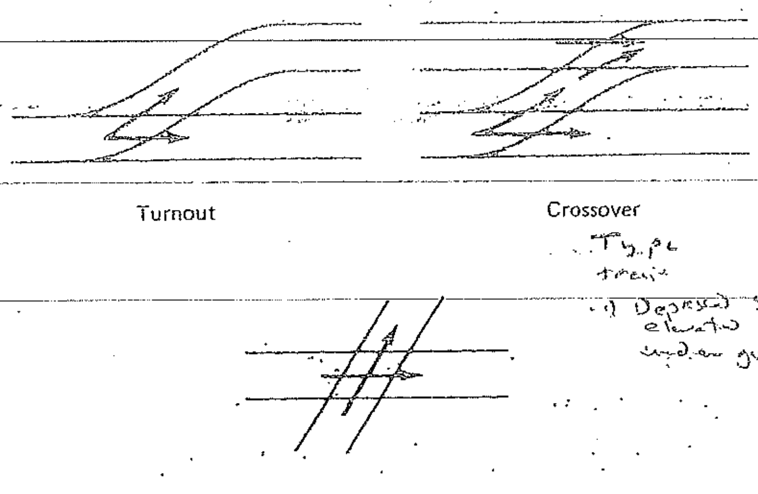
RAILROAD INTERSECTIONS

Railroad tracks intersect at *turnouts, crossovers,* and *crossings.* Turnouts are curved sections of track that permit the diversion of rolling stock from one track to another. Where the turnout provides an intersection with another continuous parallel or nonparallel track, it is called a crossover.

At crossings, tracks intersect, permitting movement of the rolling stock on one track across the alignment of the other. Figure 13-20 shows schematic arrangements for the various types of railroad intersections discussed in the following sections.

13-25. SWITCHES

The part of a turnout that determines the diversion of rolling stock movement through a turnout is known as a switch, although in U.S. practice the terms are often used as synonyms. A switch is designated as a left-hand or right-hand switch depending on the direction of diversion of the rolling stock into the turnout. Switches are relatively simple devices principally composed of movable rails or "points," rods that hold the points in their proper position and relationship, gauge and switchplates that support the points at their proper elevation, and heel blocks that effect a rigid joint at the head of the switch.



Type
train
1) Depressed system
elevation system
underground

Figure 13-20 Schematics of simple turnout, crossover, and crossings.

Various types of switches are available, only one of which is of standard use in the United States:

1. Split switches are the standard switch on American railroads. They have proven safe for very high speed movements. Switching is carried out by the use of one mainline rail and one turnout rail for switch rails, as shown in the schematic diagram in Fig. 13-21.
2. Stub switches are used to some extent in industrial tramways and on narrow-gauge lines with light rail. In a stub switch the track rails are physically bent by the switch mechanism. Stub switches are not considered safe for high-speed train movements.

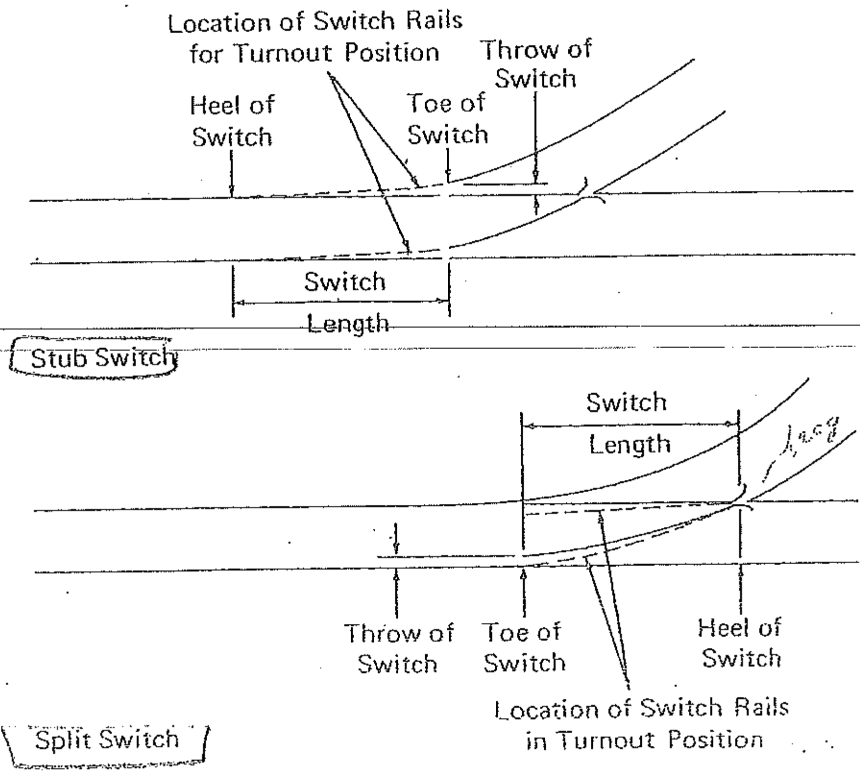


Figure 13-21 Schematics of stub and split switches.

Therefore, their use is extremely limited. They are found in a few switch yards and other low-speed areas.

3. Tongue switches are used in paved street locations. Designed for slow-moving traffic, they consist of a movable tongue on one side of the track with either another movable tongue or a fixed mate on the other side. They were once common on electric street railways.
4. Spring switches are a type of split switch where the points are spring loaded to permit point movement to allow the passage of the trailing wheels through the switch in the reverse direction of movements prevented by the points for facing movements. After the passage of the trailing wheels, the points are moved back into position by spring devices.

The arrangement of a typical left-hand turnout is shown in Fig. 13-22.

13-26. FROGS

The turnout frog is a device that permits rolling stock wheels on one rail to cross the rail of a diverging track. It performs two functions, supporting the wheel over the intersection of the flangeways and providing continuous channels for the wheel flanges. Two principal frog types are in common use, the rigid frog and the spring frog. Figure 13-23 shows typical arrangements of rigid and spring frogs. In the rigid frog, both flangeways are always open and both wing rails are bolted to the body of the frog. In the design of the conventional spring frog, the main flangeway only is always open. The turnout direction is opened when one wing moves, establishing a flangeway for the wheels.

Frogs normally are designated by a frog number, which is defined as one-half the cotangent of one-half the frog angle or the ratio of the spread at any point to the length of a bisecting line between that point and any theoretical point of frog:

$$n = \frac{1}{2} \cot \left(\frac{1}{2}\phi\right) \tag{13-3}$$

where

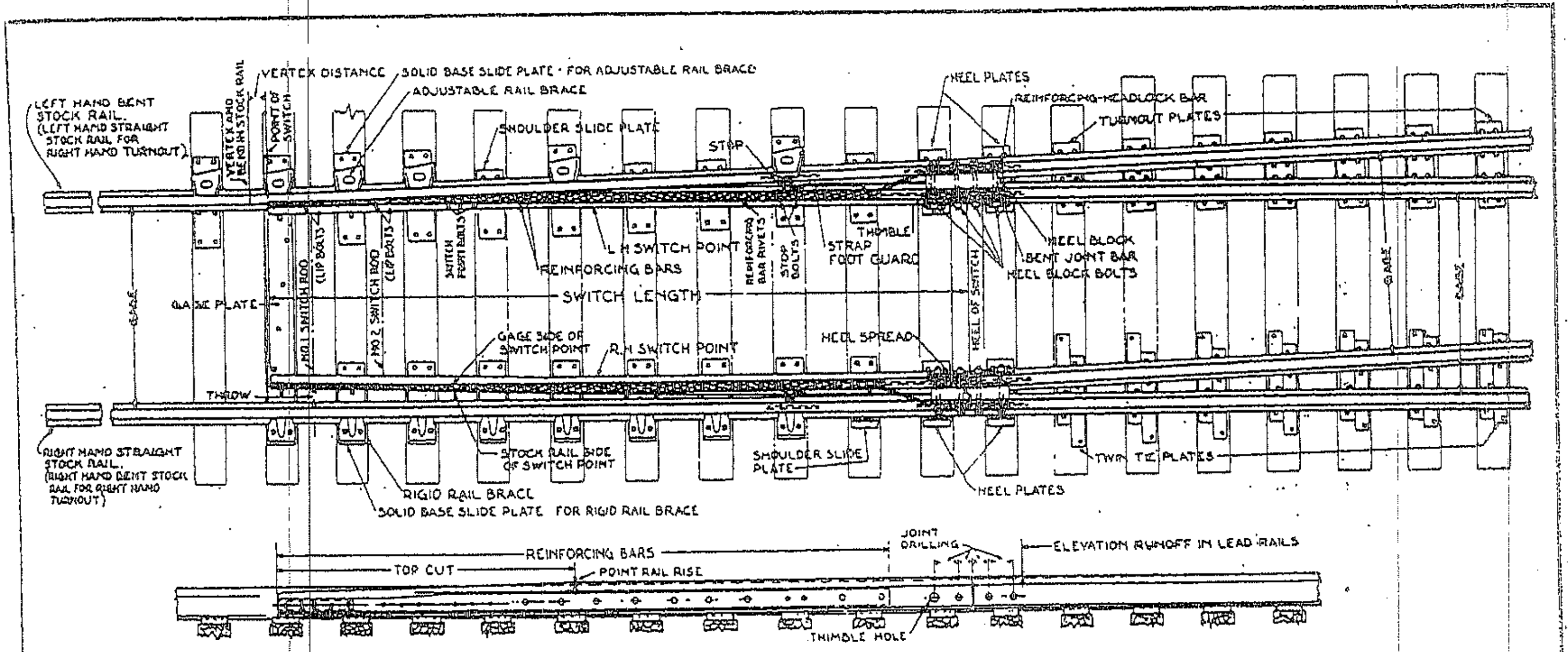
n = frog number
ϕ = frog angle

Frog angles in common use in railwork are between 9° 32′ (No. 6 frog) and 2° 52′ (No. 20 frogs). No. 18 and No. 20 frogs are in standard use on large railroad systems with mainline tracks. Slower movements can be accommodated with No. 10 and No. 12 frogs while No. 6 and No. 8 frogs are in use in sidings and industrial tramways.

13-27. CROSSINGS

Where two tracks intersect and cross, specially designed and fabricated crossings are necessary. Since the angle of intersection is usually nonstandard, specially designed frogs are necessary. Crossings can be designated into four general classifications:

1. Bolted rail crossings: All members are heat treated or open-hearth rails bolted together.
2. Manganese steel insert crossings: Manganese steel cast inserts, which have high impact resistance, are fitted into rolled rails to form the wing and points of frogs.
3. Solid manganese steel crossings in which each frog is a single solid casting.
4. Double slip switches with movable point crossings.



A.R.E.A. Preferred Names for Parts of Split Switches

Figure 13-22 A typical split switch left-hand turnout. (Courtesy American Railway Engineering Association.)

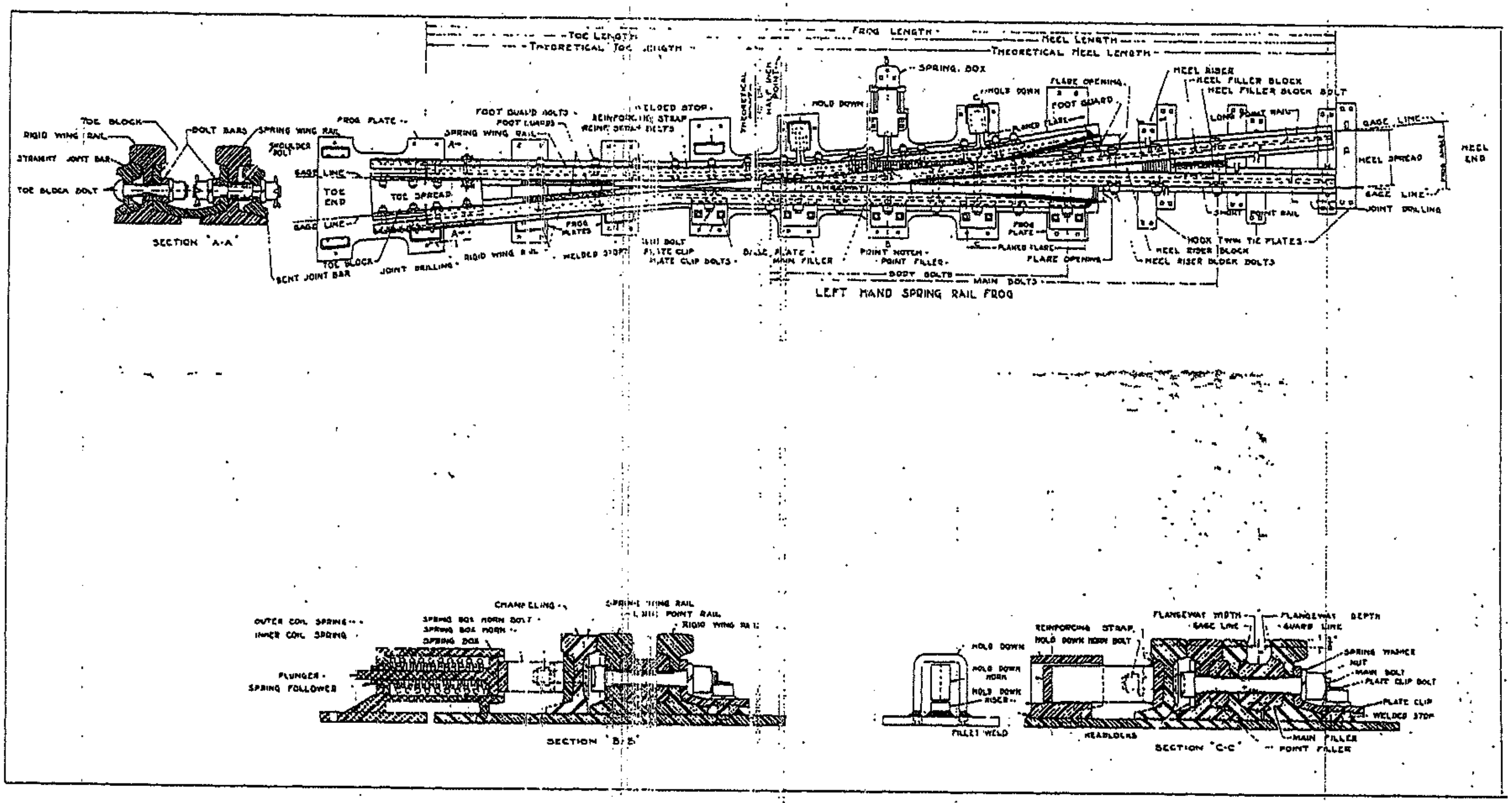


Figure 13-23 Rigid and spring frog arrangements. (Courtesy American Railway Engineering Association.)

The choice of crossing type is dependent on speeds of operation, the angle of intersection, and the degree of curvature of the track.

13-28. RAILROAD GRADE INTERSECTIONS

In the design of a highway that intersects a railroad at grade, consideration must be given to approach grades, sight distance, drainage, volume or vehicular traffic, and the frequency of regular train movements at the particular intersection. The particular type of surfacing and kind of construction at railroad crossings at grade will depend upon the class of railroad and kind of roadway improvement.

All railroad intersections at grade require proper advance warning signs. At crossings on heavily traveled highways where conditions justify, automatic devices should be installed. Recommended standards for railroad–highway–grade crossing protection have been adopted by the Association of American Railroads.

The use of grade separations at railroad crossings is recommended at all mainline railroads that consist of two or more tracks and at all single-line tracks when regular train movements consist of six or more trains per day. Other considerations for separating railroad and highway traffic are the elements of delay and safety.

Railroad grade separation structures may consist of an overpass on which the highway is carried over the railroad or an underpass that carries the highway under the railroad. The selection of the type of structure will depend in large part upon the topographical conditions and a consideration of initial cost. Drainage problems at underpasses can be serious. Pumping of surface and subsurface water may have to be carried on a large part of the time, and the failure of power facilities sometimes causes flood conditions at the underpass with the resultant stoppage of traffic.

PROBLEMS

1. A highway is to be built in a suburban environment with level terrain. The forecast AADT is 9500 vehicles/day with 12 percent trucks. The directional factor is 0.55, and the peak-hour factor is 0.90. The desired level of service is C. Assume that the K factor is 0.18 and the free-flow speed is 96 km/hr (60 mph). Determine the number of lanes that will be needed in each direction.
2. Two intersecting highways cross at an angle of 80 degrees. Make a sketch of a cloverleaf interchange for complete traffic flow.
3. Discuss the different philosophies that form the basis for the radically different design for a local residential subdivision street and an expressway.
4. How does the U.S. railroad gauge and other appurtenances compare with those of European countries? Why are concrete ties used extensively in Europe but not in the United States?
5. Suppose a subdivision developer has a 3500-ft by 1200-ft rectangular piece of property that has streets abutting on two of the adjacent sides. Sketch a street layout and residential lot layout plan using the design criteria given here:
   a. Minimum lot size, ⅓ acre
   b. Maximum lot size, ½ acre
   c. Minimum lot frontage, 100 ft
   d. Minimum set-back of houses from right-of-way line, 50 ft
6. Suppose a subdivision developer has a 1200-m by 500-m rectangular piece of property

that has streets abutting on two of the adjacent sides. Sketch a street layout and residential lot layout plan using the design criteria given here:

a. Minimum lot size, 0.2 hectare
b. Maximum lot size, 0.3 hectare
c. Minimum lot frontage, 35 m
d. Minimum set-back of houses from right-of-way line, 18 m

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Chapter 14

Design of Streets, Highways, and Railways: Drainage, Earthwork, and Pavements

Three important aspects of railway and roadway design will be discussed in this chapter: the provision of adequate drainage, earthwork operations, and pavements.

DESIGN OF DRAINAGE STRUCTURES AND FACILITIES

In this chapter, the discussion of street, highway, and railway drainage will be limited to a discussion of surface drainage, which is the process of controlling and removing excess water from the traveled way.

Three major topics will be discussed in this section:

1. Estimation of runoff
2. Hydraulic design of culverts and drainage ditches
3. A comparison of alternative drainage systems

The first of these topics deals with techniques for estimating the quantity of runoff water on the bases of certain rainfall data and land use characteristics. Culvert design essentially encompasses a discussion of nonuniform open channel flow, while the design of drainage ditches involves uniform open channel flow. Approaches used in the design of underground storm drainage systems such as those used for city streets are given in Section 19-7.

In the sections that deal with hydraulic design, emphasis will be placed on the use of empirical data rather than on fluid flow theory. For the reader who is more interested in theoretical approaches to drainage design, reference should be made to one or more of the numerous textbooks in fluid mechanics.

14-1. SURFACE DRAINAGE

Before proceeding with a discussion of the techniques for estimating runoff quantities, it should first be noted that consistent with other design objectives, every effort should be made to remove precipitation from the traveled way as expeditiously as possible. Water that is not removed quickly from a highway or railway nearly always is harmful to the load-carrying capability of the pavement system. Furthermore, floodwaters serve as a deterrent to free traffic movements and create unnecessary perils for the users of the facility. Uncontrolled water movements may weaken, damage, or even destroy transportation structures and the pavement system. For these reasons, highway designers provide pavement crown and shoulder slopes to expedite the removal of surface water.[1] Similarly, railroad design engineers specify an open-graded ballast material and a sloped subgrade to ensure adequate and quick drainage. Well-designed culvert and bridge structures must be provided for railways and highways alike to prevent destructive back waters and roadway overtopping from occurring.

To ensure adequate drainage, unpaved side ditches should have a slope of at least 0.5 percent. Experience has shown that paved ditches with an average slope of about 0.3 percent will drain satisfactorily.

14-2. HYDROLOGIC APPROACHES AND CONCEPTS

Several approaches have been used to estimate the quantity of runoff for drainage design. When the drainage structure is to handle the flow of an existing stream, as in the case of some culverts and most bridges, historical records may be available for drainage design. For such "gauged" sites, records of stream flow can be analyzed statistically to provide an estimated peak design flow for a given "return period."

The term *return period* refers to the estimated frequency of occurrence of the design storm. For example, if the system is designed for a return period of 10 years, the statistical assumption is that the system will accommodate the most severe storm that will occur once in 10 years. Obviously, the selection of a return period of 50 years instead of 10 years would mean designing for a more severe storm and usually a more costly system. On the other hand, if the return period is 5 years, the intensity of the design storm will be less, resulting in most cases in a less costly system. Economic losses from more frequent failure of the system, however, might offset any savings in construction costs.

The choice of a frequency of occurrence of the design storm is largely a matter of experience and judgment, but it may be established by policy of the agency, company, or professional association. For example, the American Railway Engineering Association [1] recommends that culverts be designed to discharge as a minimum:

1. A 25-year flood without static head at the entrance
2. A 100-year flood using the available head at the entrance, the head to 2-ft below the base of the rail, or the head to a depth of 1.5 times the diameter/rise, whichever is less

Stream gauge records for particular regions have also been used to develop statistical regression equations for many areas of the country [2]. With basic watershed data such as the drainage area and the average slope of the stream, these equations can be used to estimate peak design flows for ungauged sites within the hydrologic region.

Useful information may be gained from existing structures and the natural stream. Drainage structures above and below the proposed location may provide information on the size and types of installations that have given satisfactory service on the same stream.

[1]Cross-sectional design criteria to ensure good drainage are given in Chapter 13.

~~An examination of the natural channel at the site may also show evidences left by flood~~ crests that have occurred in the past, and an estimate can be made of the quantity of water that has been carried by the stream during flood periods.

Of course, stream flow records are not often available, and a detailed physical examina~~tion of the existing channel may not be feasible. These conditions commonly prevail,~~ in fact, for small channels that drain only a few hundred hectares (acres). A number of formulas and analytical procedures have been developed for estimating runoff from small drainage areas, the most widely used one being the rational method. Before we describe this formula, a brief discussion of the factors that influence the magnitude of surface runoff will be given.

14-3. FLOOD HYDROGRAPHS

A flood wave passing a point along a stream can be represented by a figure known as a flood hydrograph. See Fig. 14-1. The flow increases to a maximum, then recedes. Engineers have traditionally designed highway and railway drainage structures to accommodate the peak flow. Reduction of peak flows can be achieved by the storage of water that falls on the site in detention basins, storm drainage pipes, and swales and ditches. Storage has the effect of broadening and flattening the flood hydrograph and decreasing the potential for downstream flooding. With storage, storm water is released to drainage structures and the stream at a reduced rate of flow. This concept should be considered for use in highway and railway design where existing downstream drainage structures are inadequate to handle peak flow rates or where the facility would contribute to increased peak flow rates and aggravate downstream flooding problems [3]. Additional information on storage routing procedures is given in references 2 and 4.

14-4. COEFFICIENT OF RUNOFF

Runoff results from precipitation that falls on the various surfaces of the watershed. A part of the precipitation evaporates and some of it may be intercepted by vegetation. A portion of the precipitation may infiltrate the ground or fill depressions in the ground

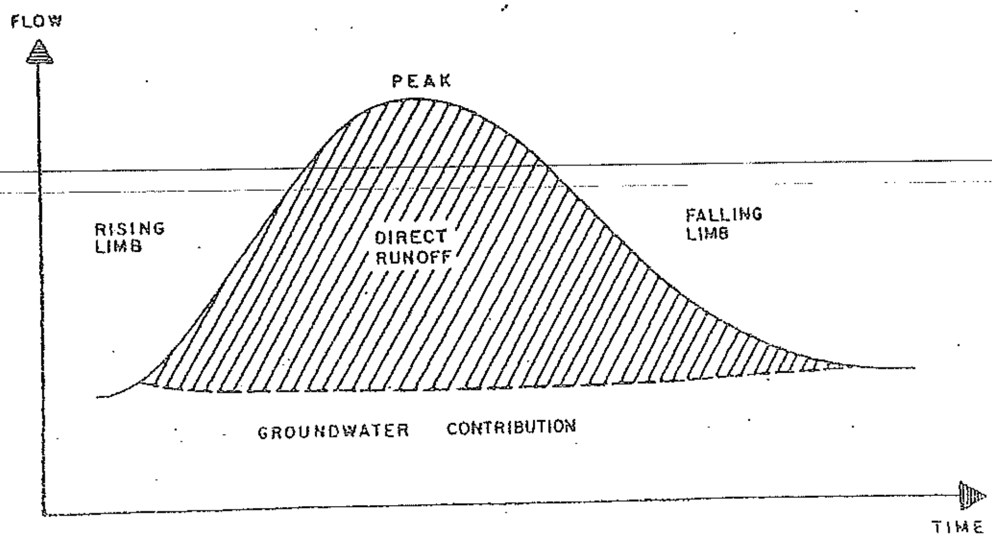


Figure 14-1 A flood hydrograph. (Courtesy Federal Highway Administration.)

Table 14-1 Coefficients of Runoff to be Used in the Rational Formula

Type of Drainage Area	Coefficients of Runoff, C
Rural areas[a]	
Concrete or asphalt pavement	0.8–0.9
Asphalt macadam pavement	0.6–0.8
Gravel roadways or shoulders	0.4–0.6
Bare earth	0.2–0.9
Steep grassed areas (2:1)	0.5–0.7
Turf meadows	0.1–0.4
Forested areas	0.1–0.3
Cultivated fields	0.2–0.4
Urban areas	
Flat residential, with about 30% of area impervious	0.40
Flat residential, with about 60% of area impervious	0.55
Moderately steep residential, with about 50% of area impervious	0.65
Moderately steep built-up area, with about 70% of area impervious	0.80
Flat commercial, with about 90% of area impervious	0.80

[a]For flat slopes or permeable soil, use the lower values. For steep slopes or impermeable soil, use the higher values.

surface. The storm runoff for which roadside ditches and drainage structures must be designed, then, is the precipitation minus the various losses that occur.

These losses and thus the runoff are strongly dependent upon the slope, vegetation, soil condition, and land use of the watershed. The designer should remember that certain of these factors, notably vegetation and land use, will not remain constant over time. For example, a drainage structure that is designed to accommodate a watershed used for agricultural purposes may prove totally inadequate if the land is later developed into a residential subdivision.

Most analytical procedures for estimating runoff involve the use of a coefficient of runoff to take into consideration the hydrologic nature of the drainage area. Values of the coefficient of runoff for one method of estimating the runoff quantity are given in Table 14-1. If the drainage area under consideration consists of several land use types, a coefficient should be chosen for each such subarea. The runoff coefficient for the entire area then should be taken as the weighted average of the coefficients for the individual areas.

14-5. RAINFALL INTENSITY, DURATION, AND FREQUENCY

Rainfall intensity is the rate at which rain falls, typically expressed in millimeters per hour or inches per hour. Because of the capriciousness of weather, it is necessary to discuss rainfall intensity in the context of rainfall frequency and duration.

Rainfall intensity–duration data have been collected and published by the National Weather Service [5] for various sections of the United States. Typical rainfall intensity–duration curves are shown in Fig. 14-2 for return periods of 2, 5, 10, 25 and 50 years. As can be seen from the figure, rainfall intensity varies greatly with the duration of rainfall. The average rainfall intensity for short periods of time is much greater than for

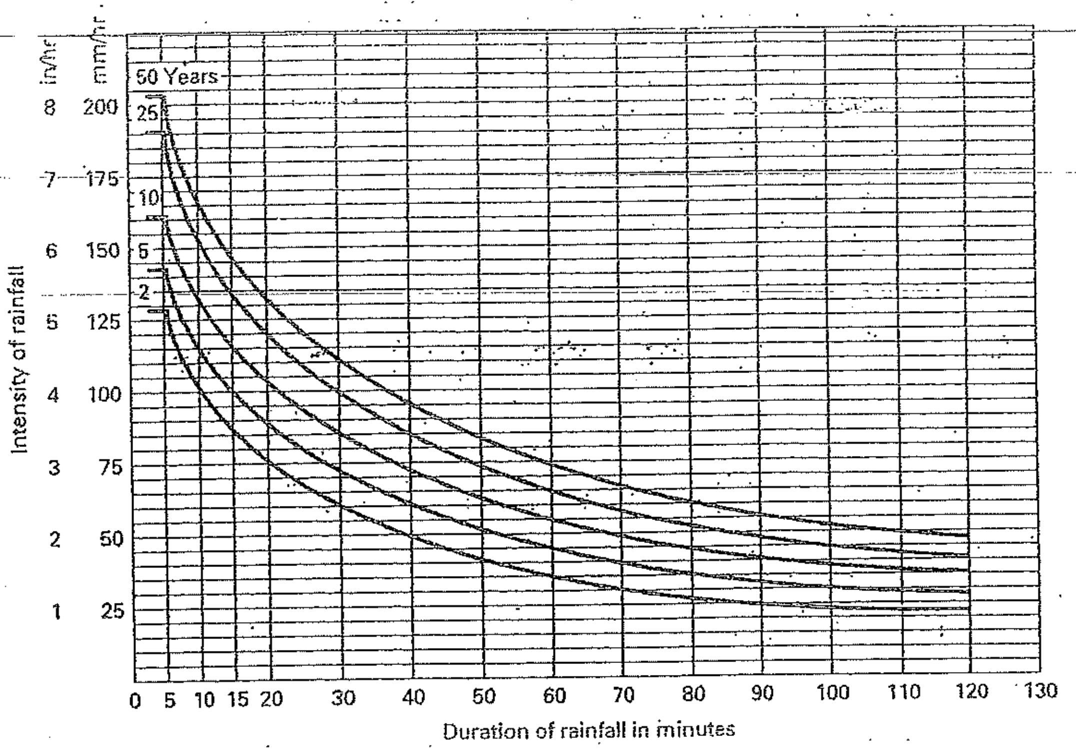


Figure 14-2 Typical rainfall intensity-duration curves. (Courtesy Federal Aviation Administration.)

long periods. In the design of most railroad and highway drainage facilities, a duration equal to the *time of concentration* is chosen.

14-6. TIME OF CONCENTRATION

The time of concentration is defined as the time required for a particle of water to flow from the most remote point in the drainage area to the outlet end of the drainage structure. In other words, it consists of the time of overland flow plus the time of flow in the drainage system. For most railroad and highway culverts, the time of flow in the drainage system is negligible in comparison to the time of overland flow. Thus, for the purpose of this chapter, the terms *time of overland flow* and *time of concentration* may be used synonymously.

Time of concentration varies with the land slope, type of surface, rainfall intensity, size and shape of the drainage area, and many other factors. A number of empirical studies have been made relating time of concentration to the slope and dimensions of the drainage area, for example, reference 6. When the particular drainage area consists of several types of surfaces, the time of overland flow must be determined by summing the respective times computed for flow along the various surfaces from the most remote point to the inlet.

14-7. THE RATIONAL METHOD

By far, the most popular method for estimating runoff from small drainage areas is the rational method. In the rational formula, the quantity of water falling at a uniform rate is related by simple proportion to the total quantity that appears as runoff:

$$Q = CIA \qquad (14\text{-}1)$$

where

Q = runoff, ft³/sec

C = a coefficient representing the ratio of runoff to rainfall (typical values of C given in Table 14-1)

I = intensity of rainfall, inches per hour for the estimated time of concentration

A = drainage area in acres (may be determined from field surveys, topographical maps, or aerial photographs)

In metric units, the equation becomes

$$Q\ (\text{m}^3/\text{sec}) = 0.0028CIA \qquad (14\text{-}2)$$

where I is expressed in millimeters per hour and A in hectares.

The designer should contact the nearest office of the National Weather Service for rainfall intensity data. In the event that suitable local rainfall intensity data are not available, approximate data may be obtained from Fig. 14-3, a chart published by the Federal Highway Administration. This map shows rainfall intensity values in inches per hour for various areas of the contiguous United States for a two-year, 30-min rainfall. The two-year rainfall intensity for other durations may be obtained by multiplying by the following factors:

Rainfall Duration (min)	Factor
5	2.22
10	1.71
15	1.44
20	1.25
40	0.80
60	0.60
90	0.50
120	0.40

To obtain rainfall intensities corresponding to their intervals of recurrence, the values of Fig. 14-3 should be multiplied by the following factors:

Recurrence Interval (years)	Factor
1	0.75
2	1.00
5	1.30
10	1.60
25	1.90
50	2.20

14-8. COMPUTER SOFTWARE IN HYDROLOGY [7]

The Federal Highway Administration has sponsored the development of an integrated system of seven hydrology and hydraulic computer programs in flood computation and drainage design known as HYDRAIN [8]. The software requires a minimum equipment component of MS DOS 3.0 or better, IBM XT, 640K RAM, hard disk, and monochrome monitor. The HYDRAIN software is available through FHWA distribution agents, including McTrans Center, University of Florida, and PCTrans, University of Kansas.

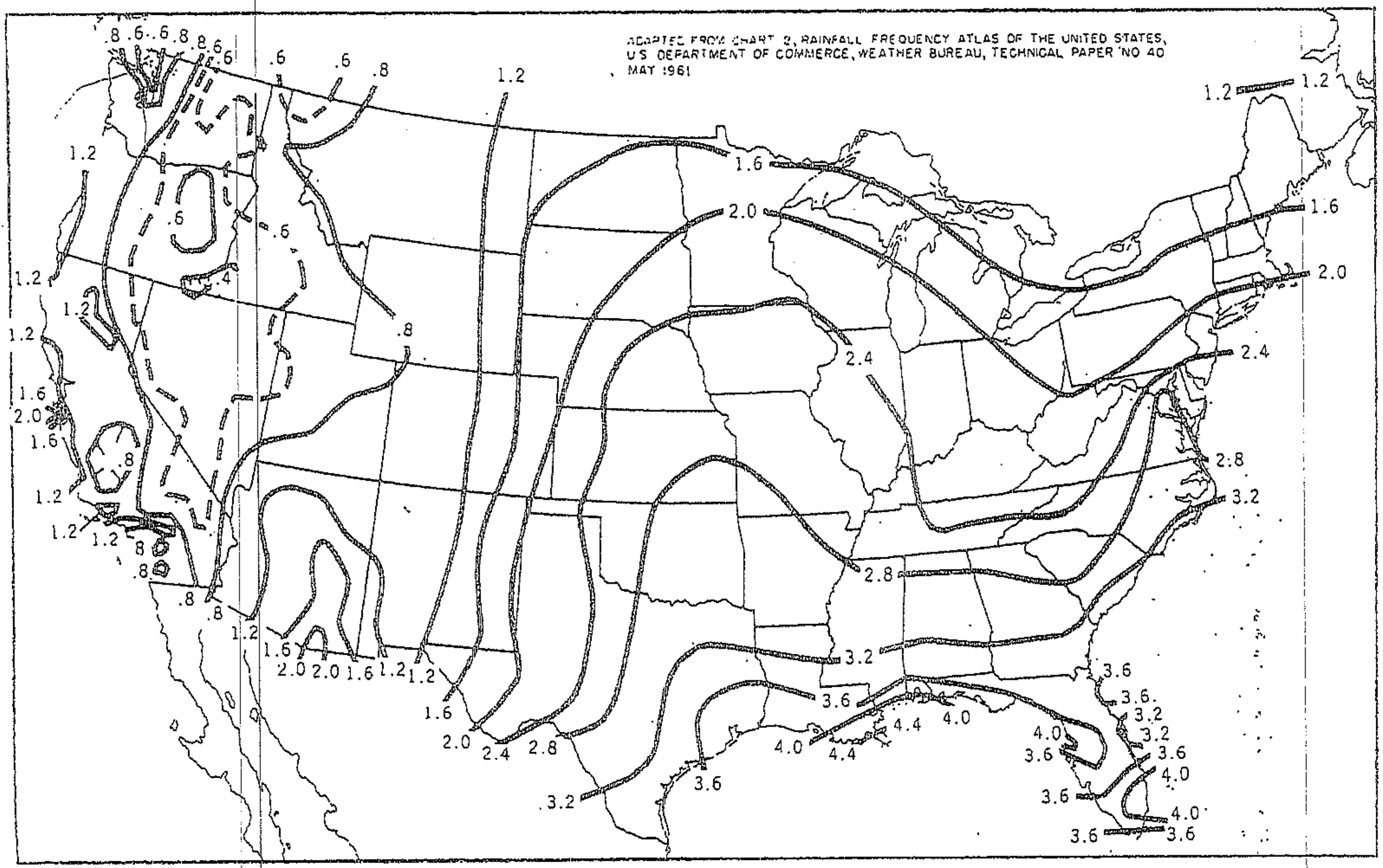


Figure 14-3 Map of contiguous United States showing two-year, 30-min rainfall intensity. (Source: *Design of Roadside Drainage Channels*, Federal Highway Administration, 1965.)

One of the programs included in the integrated package is a hydrology program known as HYDRO. This system is a computer-based subset of Hydraulic Engineering Circular No. 19 [9] and includes some areas of Hydraulic Engineering Circular No. 12 [10].

Users of HYDRO are able to:

1. Estimate a time of concentration and intensity of rainfall for any site in the continental United States based on location and basin characteristics.
2. Create a duration-versus-rainfall-intensity curve for any user provided frequency and location in the continental United States.
3. Determine a peak flow using the rational method.
4. Determine a peak flow using regression equations developed by state and federal agencies.

HYDRAULIC DESIGN OF CULVERTS AND DRAINAGE CHANNELS

In the preceding paragraphs we have discussed concepts and design procedures relating to the estimation of the quantity of runoff from a drainage basin. The following sections will deal with principles and techniques for the hydraulic design of culverts and other drainage structures for streets, highways, and railways. Some of the applicable fundamental principles govern fluid flow in conduits flowing under pressure and in open channels. This material admittedly is sketchy. For the reader who wishes a more extensive treatment of this subject, many transportation agencies have published drainage manuals, for example, reference 10.

Following a brief review of hydraulic principles, various types of culvert flow will be discussed, after which typical design charts for culverts and open channels will be introduced and described.

14-9. FUNDAMENTAL PRINCIPLES: CONDUITS FLOWING FULL

Two fundamental principles form the basis for the theory of conduits accommodating fluid flow under pressure: the conservation of mass (expressed as the continuity equation) and the conservation of energy (expressed as the Bernoulli equation).

The continuity equation merely states that the quantity of flow throughout a given flow system is constant:

$$Q = va \tag{14-3}$$

where

Q = rate of flow, m³/sec (ft³/sec)
v = average velocity of flow, m/sec (ft/sec)
a = cross-sectional area, m² (ft²)

The Bernoulli equation states that the total energy (pressure, kinetic, and potential) at a selected section of a flow system is equal to the energy at some previous section provided allowance is made for any energy added to or taken from the system:

$$z_1 + \frac{P_1}{w_1} + \frac{v_1^2}{2g} - \Sigma H_L = z_2 + \frac{P_2}{w_2} + \frac{v_2^2}{2g} \tag{14-4}$$

where

P/w = pressure energy

$\frac{v^2}{2g}$ = kinetic energy

z = potential energy

ΣH_L = summation of energy losses (head losses)

The head losses ΣH_L usually are expressed in terms of velocity of flow. These losses are due primarily to friction losses within the conduit and entrance losses.

Friction Losses. Friction losses in a conduit flowing full are most commonly obtained by the Darcy–Weisbach equation:

$$H_f = f\frac{L}{D}\frac{v^2}{2g} \tag{14-5}$$

where

H_f = frictional losses, m (ft)
f = friction factor
L = length of pipe, m (ft)
D = diameter of pipe, m (ft)
$v^2/2g$ = velocity head, m (ft)

The friction factor is a dimensionless measure of pipe resistance that depends on the characteristics of the pipe and the flow. For culverts the friction factor is most commonly given in terms of the coefficient of the Manning equation, n. (See Table 14-2.) The relationship between n and f is given by the following equations:

$$f = 125\frac{n^2}{D^{1/3}} \quad \text{(metric system)} \tag{14-6}$$

$$f = 185\frac{n^2}{D^{1/3}} \quad \text{(traditional U.S. units)} \tag{14-7}$$

where

D = diameter, m (ft)

Entrance Losses. Another major source of head loss occurs when flow is constricted in a culvert entrance. Entrance losses are due mainly to the expansion of flow following the entrance constriction. These losses may be computed by multiplying the velocity head of the full pipe times a constant value called the entrance loss coefficient K_e:

$$H_L = K_e\frac{v^2}{2g} \tag{14-8}$$

Entrance loss coefficients vary widely with different types of entrance geometry. Typical values of K_e for various types of entrances are given in Table 14-3.

For circular culverts flowing full, the difference in elevation between the upstream and

Table 14-2 Value of Manning's Roughness Coefficient

Type of Channel or Structure	Values of n
Open channels for type of lining shown	
Smooth concrete	0.013
Rough concrete	0.022
Riprap	0.03–0.04
Asphalt, smooth texture	0.013
Good stand, any grass; depth of flow more than 6 in.	0.09–0.30
Good stand, any grass; depth of flow less than 6 in.	0.07–0.20
Earth, uniform section, clean	0.016
Earth, fairly uniform section, no vegetation	0.022
Channels not maintained, dense weeds	0.08
Natural stream channels (surface width at flood stage is 100 ft)	
Fairly regular section	
Some grass and weeds, little or no brush	0.030–0.035
Dense growth of weeds, depth of flow materially greater than weed height	0.035–0.05
Some weeds, light brush on banks	0.035–0.05
Some weeds, heavy brush on banks	0.05–0.07
Some weeds, dense willows on banks	0.06–0.08
For trees within channel, with branches submerged at high stage, increase all above values by	0.01–0.02
Irregular sections with pools, slight channel meander, increase values given above by about	0.01–0.02
Culverts	
Concrete pipe and boxes	0.012
Corrugated metal	
Unpaved	0.024–0.027
Paved 25%	0.021–0.026
Fully paved	0.012

downstream water surface H (illustrated in Figs. 14-6*a* and 14-6*b*) is equal to the velocity head plus the energy lost at the entrance and in the culvert:

$$H = \left(1 + K_e + \frac{125n^2L}{D^{4/3}}\right)\frac{v^2}{2g} \quad \text{(metric system)} \tag{14-9}$$

$$H = \left(1 + K_e + \frac{185n^2L}{D^{4/3}}\right)\frac{v^2}{2g} \quad \text{(traditional U.S. units)} \tag{14-10}$$

where

L = length of culvert, m (ft)
D = culvert diameter, m (ft)
v = velocity, m/sec (ft/sec)
g = acceleration due to gravity, 9.8 m/sec^2 (32.2 ft/sec^2)

Table 14-3 Entrance Loss Coefficient K_e: Outlet Control and Full or Partly Full

Type of Structure and Design of Entrance	Coefficient K_e
Pipe: concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, square cut end	0.5
Headwall, square edge	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Pipe or pipe arch: corrugated metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls, square edge	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Box: reinforced concrete	
Headwall parallel to embankment	
Square-edged on three edges	0.5
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Side or slope tapered inlet; all culvert types	0.2

Source: Hydraulic Design of Improved Inlets for Culverts, Federal Highway Administration, Washington, DC, 1972.

14-10. FUNDAMENTAL PRINCIPLES: OPEN CHANNEL FLOW

In open channels, the water surface, which is exposed to atmospheric pressure, serves as a flow boundary. Flow in open channels must, therefore, adjust itself so that the pressure at the water surface is equal to the pressure of the atmosphere. Open channel flow exists in conduits flowing part full as well as in open drainage ditches.

The basic laws of continuity of flow and conservation of energy also underlie the analysis of flow in open channels. In this case, however, the term for pressure is eliminated from the Bernoulli equation since the flow occurs under atmospheric pressure, which is assumed to be constant. The equation becomes

$$z_1 + d_1 + \frac{v_1^2}{2g} - \Sigma H_L = z_2 + d_2 + \frac{v_2^2}{2g} \qquad \textbf{(14-11)}$$

where

z = elevation of the bottom of the channel above a horizontal datum, m (ft)
d = depth of water in the channel, m (ft)
v = average velocity of flow, m/sec (ft/sec)
g = acceleration due to gravity, 9.8 m/sec^2 (32.2 ft/sec^2)

The energy relationships for open channel flow are shown by Fig. 14.4. Uniform flow occurs when the total energy line shown in the figure is parallel to the channel slope. Uniform flow is not often attained in highway and railway culverts.

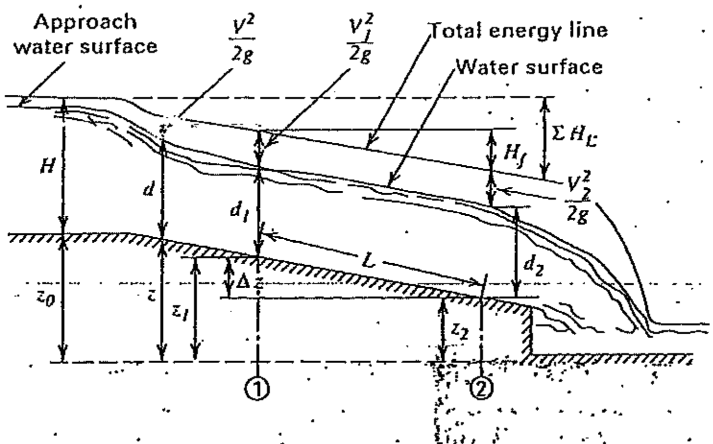


Figure 14-4 Definition sketch for open channel flow. (Source: *Handbook of Concrete Culvert Pipe Hydraulics*, Portland Cement Association, 1964.)

14-11. TYPES OF CULVERT FLOW

The type of flow occurring in a culvert depends upon the total energy available between the inlet and outlet. Naturally occurring flow is one that will completely expend all of the available energy. Energy is thus expended at entrances, in friction, in velocity head, and in depth.

The flow characteristics and capacity of a culvert are determined by the location of the control section [11]. A control section in a culvert is similar to a control valve in a pipeline. The control section may be envisioned as the section of the culvert that operates at maximum flow, the other parts of the system have a greater capacity than actually is used.

Laboratory tests and field studies have shown that highway and railway culverts operate with two major types of control: inlet control and outlet control. Examples of flow with inlet control and outlet control are shown, respectively, by Figs. 14-5 and 14-6.

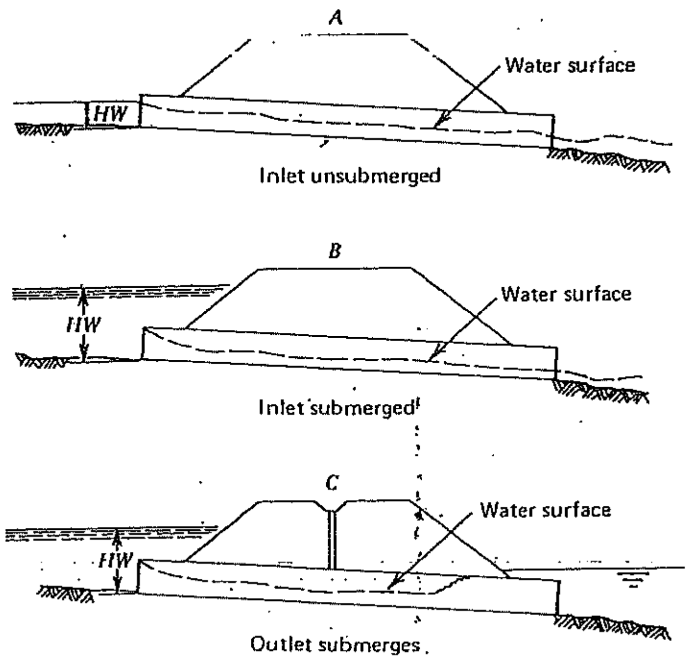


Figure 14-5 Inlet control for culverts. (Source: *Hydraulic Design for Improved Inlets for Culverts*, Federal Highway Administration, August 1972.)

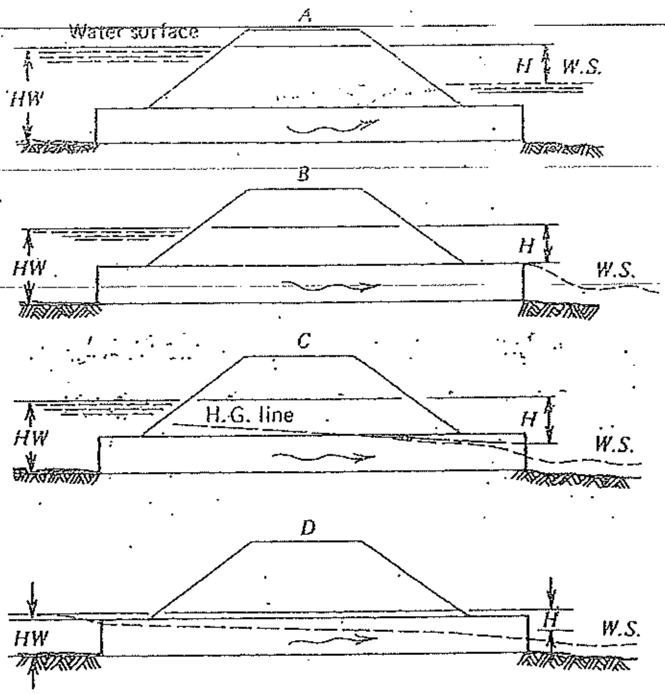


Figure 14-6 Outlet control for culverts. (Source: *Hydraulic Design for Improved Inlets for Culverts,* Federal Highway Administration, August 1972.)

Under inlet control, the discharge capacity of a culvert depends primarily on the depth of headwater at the entrance and the entrance geometry (barrel shape, cross-sectional area, and type of inlet edge). Inlet control commonly occurs when the slope of the culvert is steep and the outlet is not submerged. Such conditions are commonly found in minor watercourses in hilly terrain.

Maximum flow in a culvert operating with outlet control depends on the depth of headwater and entrance geometry and the additional considerations of the elevation of the tailwater in the outlet and the slope, roughness, and length of the culvert. This type of flow most frequently occurs on flat slopes, especially where downstream conditions cause the tailwater depth to be greater than the critical depth.

14-12. CULVERT DESIGN CHARTS

It is possible by involved hydraulic computations to determine the probable type of flow under which a given culvert will operate and to estimate its capacity. These computations may often be avoided by using design charts and nomographs published by the Federal Highway Administration. From these charts and graphs, the headwater depths for both inlet control and outlet control may be determined for practically all combinations of culvert size, material, entrance geometry, and discharge.

Hydraulic charts in the form of nomographs have been published by the Federal Highway Administration in Hydraulic Design Series No. 5 [2]. Examples of these nomographs are shown in Figs. 14-7 and 14-8. The inlet control charts are based on laboratory research conducted by the National Bureau of Standards and the U.S. Coast and Geodetic Survey. The nomographs for outlet control were prepared from computations based on fundamental energy relationships.

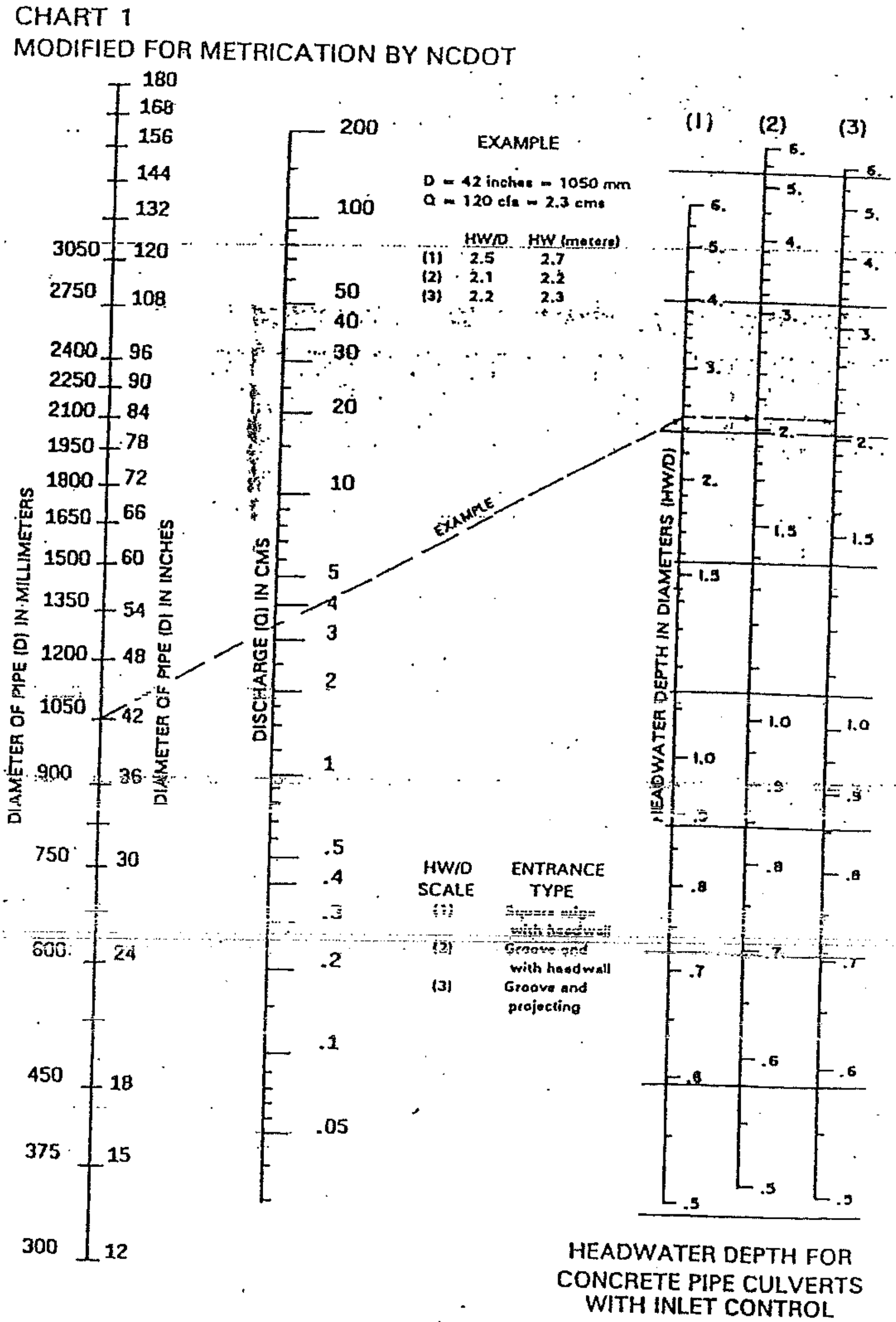


Figure 14-7 Nomograph for determining headwater depth for concrete pipe culverts with inlet control. (Courtesy Federal Highway Administration and North Carolina Department of Transportation.)

14-13. IMPROVED CULVERT INLET DESIGN

With inlet control, flow in the culvert barrel is very shallow, and the potential capacity of the barrel generally is wasted. Since the barrel is usually the most expensive component of the structure, flow under inlet control tends to be uneconomic. Surveys of culvert design practices by highway agencies indicate that millions of dollars could be saved each

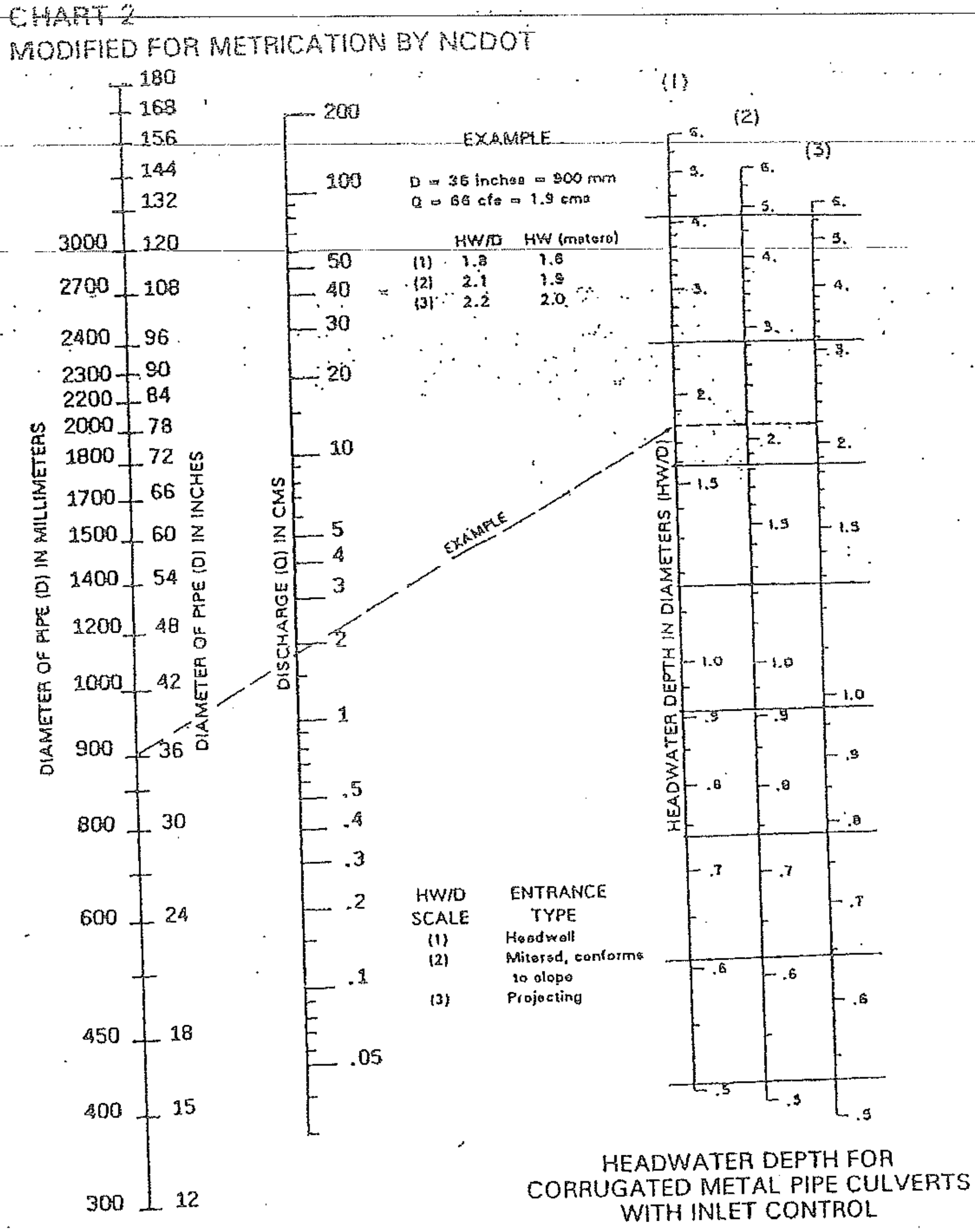


Figure 14-8 Nomograph for determining headwater depth for corrugated pipe culverts with inlet control. (Courtesy Federal Highway Administration and North Carolina Department of Transportation.)

year by the use of improved inlet design concepts. An article by Normann [12] in *Civil Engineering*, abstracted here, describes fundamental concepts of improved inlet design. For more detailed information on this important subject, the reader should refer to the FHWA Publication *Hydraulic Design of Improved Inlets for Culverts* [13].

Three basic improved inlet designs have been proposed by the FHWA: (1) bevel-edged inlets (Fig. 14-9*a*), (2) side-tapered inlets (Fig. 14-9*b*), and (3) slope-tapered inlets (Fig. 14-9*c*). These inlets improve hydraulic performance in two ways: (1) by reducing the flow contraction at the culvert inlet and more nearly filling the barrel and (2) by lowering the

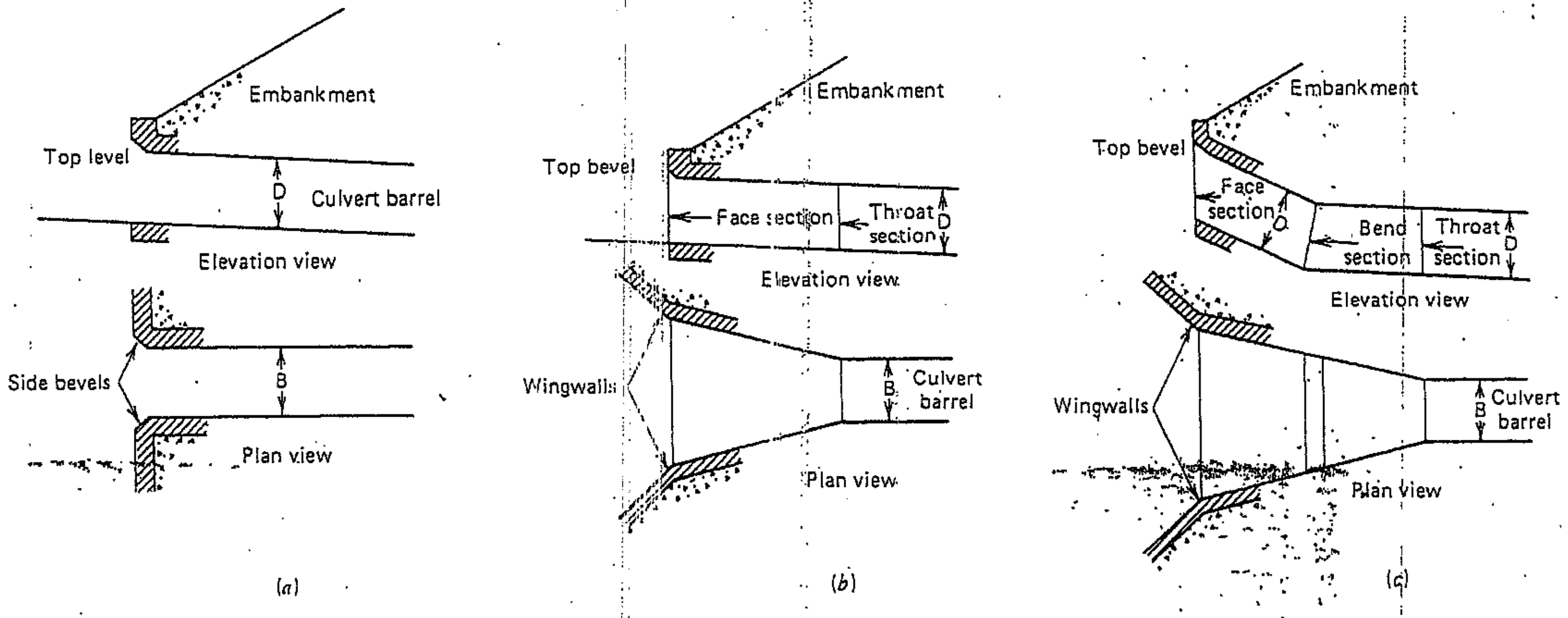


Figure 14-9 Improved culvert inlet designs. (*a*) Bevel-edge inlet. (*b*) Side-tapered inlet. (*c*) Slope-tapered inlet.

inlet control section and thus increasing the effective head exerted at the control section for a given headwater pool elevation.

Bevel-Edged Inlet. The bevel-edged inlet is the least sophisticated inlet improvement and the least expensive. The degree of improvement is recommended for use on all culverts, in both inlet and outlet control. For concrete pipe culverts, the groove end, facing upstream, will serve essentially the same purpose as the bevel-edged inlet.

Side-Tapered Inlet. The side-tapered inlet increases hydraulic efficiency by further reducing the contraction at the inlet control section, normally located near the throat section. The face section is designed to be large enough so as not to restrict the flow. The roof and floor of the inlet are straight-line extensions of the culvert roof and floor, and the tapered side walls meet the barrel walls at a smaller angle than the beveled edges (9.5 to 14 degrees versus 33 to 35 degrees). Because of the slope of the structure, the throat section is somewhat lower than the face, thus concentrating more head on the control section for a given headwater elevation.

Slope-Tapered Inlet. The slope-tapered inlet incorporates both methods of increasing hydraulic performance: reducing the entrance contraction and lowering the control section. This design provides an efficient control section at the throat, similar to that provided by the side-tapered inlet, and further increases the concentration of head on the throat. The face section remains near the streambed elevation, and the throat is lowered by incorporating a fall within the inlet structure. This fall reduces the slope of the barrel and increases the required excavation.

In addition to their use on new installations, improved inlets, especially beveled-edge and side-tapered inlets, may be added to existing barrels to increase hydraulic performance if the existing inlet is operating in inlet control. Many times this will preclude the construction of a new barrel when the existing culvert is undersized.

A FHWA survey of 66 drainage installations indicated that benefits from improved inlet structures ranged from \$500 to \$482,000, and savings of greater than \$50,000 were quite common [13].

14-14. DESIGN OF DRAINAGE CHANNELS

The simplest type of open channel flow occurs in long channels. In this case, equilibrium is established such that the energy losses due to friction are counterbalanced by the gain in energy due to slope. Discharge of this type, which is known as uniform flow, can be computed by Manning's equation:

$$Q = \frac{aR^{2/3}S^{1/2}}{n} \tag{14-12}$$

where

Q = discharge, m^3/sec
n = channel friction factor (see Table 14-2)
a = cross-sectional area of flow, m^2
R = hydraulic radius, $= \dfrac{a}{\text{wetted perimeter}}$, m
S = channel slope, m/m (ft/ft)

In conventional U.S. units, in which Q is expressed in cubic feet per second, a in square feet, and R in feet, a unit conversion factor of 1.486 is added to the numerator of the Manning equation.

The Manning equation can be solved for discharge in a given channel if the depth of flow is known. The more common problem of solving for the depth of flow corresponding to a known discharge requires repeated trials. Charts have been published, however, providing a direct solution to the Manning equation for various sizes of rectangular, triangular, circular, and trapezoidal cross sections [14].

The Manning equation most commonly applies to the underground storm drainage systems (discussed in Section 19-7) and to the design of open drainage channels.

An example of one of the channel charts available in the literature is shown in Fig. 14-10. This chart is applicable to a trapezoidal channel with 2:1 side slopes and a constant 2-ft bottom width.

Depths and velocities shown in the chart apply accurately only to channels in which uniform flow at normal depth has been established by a sufficient length of uniform channel on a constant slope when the flow is not affected by backwater.

Depth of uniform flow for a given discharge in a given size of channel on a given slope and with $n = 0.030$ may be determined directly from the chart by entering on the Q-scale and reading normal depth at the appropriate slope line (or an interpolated slope). Normal velocity may be read on the V-scale opposite this same point. This procedure may be reversed to determine discharge at a given depth of flow.

For channel roughness other than $n = 0.030$, compute the quantity Q times n and use the $Q \cdot n$- and $V \cdot n$-scales for all readings, except those that involve values of critical depth or critical velocity. Critical depth for a given value of Q is read by interpolation from the depth lines at the point where the Q-ordinate and the critical curve intersect, regardless of channel roughness. Critical velocity is the reading on the V-scale at this same point. Where $n = 0.030$, the critical slope is read at the critical depth point. Critical slope varies with n; therefore, in order to determine the critical slope for values of n other than 0.030, it is first necessary to determine the critical depth. Critical slope is then read by interpolation from the slope lines at the intersection of this depth with the $Q \cdot n$-ordinate.

EXAMPLE 14-1[2]

Suppose, for example, one wishes to determine the depth and velocity of flow in a trapezoidal channel ($n = 0.030$) with 2:1 side slopes and a 2-ft bottom width given a flow of 100 ft^3/sec and a slope of 4.0 percent. The chart is entered at $Q = 100$ ft^3/sec and a line is projected vertically until it intersects the slope line $S_o = 0.04$. At the point of intersection, the normal depth $d_n = 1.6$ ft and the corresponding normal velocity $V_n = 10$ ft/sec is read.

To find the critical depth, velocity, and slope for these conditions, the line $Q = 100$ ft^3/sec is projected upward to its intersection with the critical curve. At the point of intersection, the following values are read:

Critical depth $d_c = 2.3$ ft.
Critical velocity $V_c = 6.6$ ft/sec
Critical slope $S_c = 0.014$

The normal depth is less than the critical depth, indicating that the flow is rapid and not affected by backwater conditions.

[2]Conventional U.S. units are used in this example in order to be consistent with available design charts.

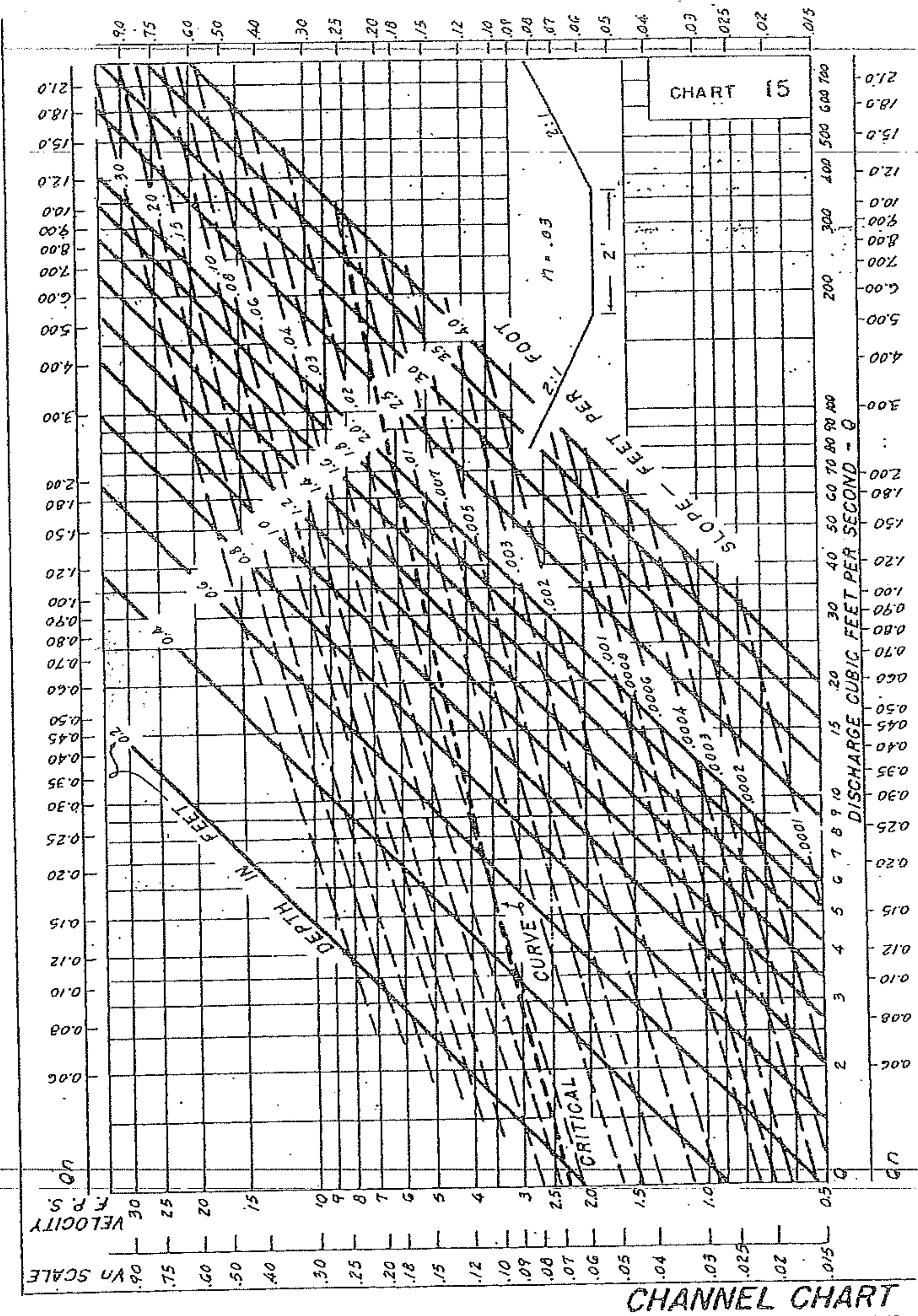


Figure 14-10 Channel chart for two-foot bottom channel with 2:1 side slopes. (Source: *Design Charts for Open Channel Flow*, Federal Highway Administration, August 1961.)

Suppose $n = 0.012$ and other conditions are the same as given above. In this case, the product $Q \cdot n$ is determined and the $Q \cdot n$ scale is used:

$$Q \cdot n = 100(0.012) = 1.2$$

Entering the $Q \cdot n$-scale at a value of 1.2 and proceeding as before yield the following values:

$$d_n = 1.2 \text{ ft} \qquad V_n = \frac{0.24}{0.012} = 20 \text{ ft/sec}$$

$$d_c = 1.5 \text{ ft} \qquad V_c = \frac{0.17}{0.012} = 14 \text{ ft/sec} \qquad S_c = 0.016$$

Erosion Control. The designer should remember that erosion of open channels is not only aesthetically offensive but may also increase roadway maintenance costs and, in its most drastic form, may create roadway hazards. There are several steps that can be taken to minimize the probability and lessen the effects of erosion. In the first place, erosion channel velocities should be avoided whenever possible. The magnitude of stream velocity that causes erosion varies, of course, with the type of channel lining. Unlined ditches in certain fine-grained soils may be eroded by streams flowing as slow as 0.5 to 0.6 m/sec (1.5 to 2.0 ft/sec), while other untreated soils may resist erosive forces caused by streams flowing up to 1.8 m/sec (6.0 ft/sec).

To avoid erosion, it may be necessary to provide protective linings in the bottom and along the sides of drainage channels. A variety of protective linings are used, including Portland cement concrete, soil-cement, rock riprap, and vegetation. To prevent erosion during the construction process, various temporary linings are utilized to protect seeded channels until vegetation has been established [15].

14-15. COMPUTER SOFTWARE FOR HYDRAULIC DESIGN [7]

The HYDRAIN system of computer programs (mentioned in Section 14-8) includes the following software to facilitate the hydraulic design of channels and culverts:

1. HYCLV analyzes and designs circular, elliptical, rectangular, and other culvert shapes. It makes it possible to compare hydraulic characteristics of different culvert types for a particular site. It ranks entrance edge conditions from most efficient to least efficient and computes exit velocities. With the software, it is possible to develop performance curves of both individual culverts and an entire system.
2. HY8 is a BASIC program that allows the user to investigate the hydraulic performance of a culvert system and considers the hydrologic inputs, storage, and energy dissipation devices and strategies. This program automates the methods presented in Hydraulic Design Series No. 5 [2], Hydraulic Engineering Circular No. 14 [16], and Hydraulic Engineering Circular No. 19 [4].
3. HYCHL provides guidance for the design of stable roadside channels and the analysis of the performance capabilities of various channel linings. This software also assists in the design of irregular channel riprap linings.

14-16. CULVERT SELECTION

While a variety of materials have been utilized, the large majority of culverts in common use fall into two classes:

1. Reinforced concrete culverts
2. Corrugated metal culverts

Reinforced concrete culverts fall into two general classes: pipes and box culverts. Concrete pipes come in a variety of diameters ranging, from 300 to 3600 mm (12 to 108 in.). Box culverts usually are formed and poured in place with rectangular or square cross section.

Concrete culverts are durable and able to withstand large stresses imposed by heavy wheel loads. In many localities, concrete culverts cost less than comparable sizes of corrugated metal culverts. Possessing a smaller roughness coefficient, concrete culverts are more efficient hydraulically than corrugated metal culverts. On the other hand, concrete culverts are heavy and not installed easily, particularly in cases involving steep slopes and large sizes.

Corrugated metal culverts are made of galvanized steel of varying thicknesses. These culverts are manufactured in diameters from 200 to 2440 mm (8 to 96 in.). Corrugations, typically 68 mm (2⅔ in.) from crest to crest and 13 mm (½ in.) deep, are formed in the sheet metal for added strength.

Corrugated metal culverts are easy to handle and install, even in large sizes and steep slope. Where very large pipe sizes are required, corrugated metal culverts may be formed from heavy plates that are bolted together at the field site.

Corrugated metal culverts may be purchased as pipe arches, the vertical dimension (rise) being about 0.6 the horizontal dimension (span). Arch culverts are used commonly and advantageously in low-fill areas where the headroom is limited. Although corrugated metal culverts are subject to deterioration in certain severe-exposure conditions, this may be largely overcome by the use of a pipe in which the invert has been covered with a heavy bituminous mixture.

EARTHWORK OPERATIONS

14-17. INTRODUCTION

Practically all highway and railroad construction jobs involve a considerable amount of earthwork. Earthwork operations in general are those construction processes that involve the soil or earth in its natural form and that precede the building of the pavement structure itself. These processes may include everything that pertains to the grading and drainage structures, which will also include clearing and grubbing, roadway and borrow excavation, the formation of embankments, and the finishing operations for the preparation of the highway or runway pavement or railroad ballast. Any or all of these construction processes may be performed on a given project and they may overlap to some extent.

Clearing is the removal of trees, shrubs, brush, and so on, from within designated areas, while grubbing refers to the removal of roots, stumps, and similar obstacles to a nominal depth below the existing ground surface. Frequently, clearing and grubbing comprise a single contract item and may include the removal of topsoil to a shallow depth. Excavation refers to the removal of earth from its natural resting place to a different place for the highway or railroad foundation. Embankments required in construction usually are formed in relatively thin layers or lifts of soil and compacted to a high degree of density. Such embankments are called rolled-earth embankments or rolled-earth fills. Hydraulic fills also may be required in construction.

Finishing operations include such items as trimming and finishing of slopes and the fine grading operations required to bring the grade to the desired final elevation.

Broadly speaking, earthwork operations may include all the operations involved in bringing the foundation to the point where the surface or ballast is to be applied.

14-18. EARTHWORK EQUIPMENT

In modern practice earthwork operations are accomplished largely by the use of highly efficient and versatile machines. Machines have been developed that are capable of performing every form of earthwork operation efficiently and economically. These include tractors, bulldozers, scraper units, shovel crane units for the excavation and moving of earth, and the sheepsfoot, pneumatic-tired, and steel-wheel rollers for compacting the earth as well as vibratory compactors and pneumatic tampers. Motor graders, trucks, and other specialized equipment may be used for earthwork operations.

A more detailed discussion of earth-moving equipment may be found in reference 7.

14-19. EXCAVATION

Excavation may be classified into four types: (1) rock excavation, (2) unsuitable excavation, (3) borrow excavation, and (4) common excavation. Rock excavation includes boulders that may be 1.5 m^3 (2 yd^3) or more in volume and all hard rock that has to be removed by blasting. In some cases, the requirement relative to the removal by blasting is not specified and a phrase such as "solid well-defined ledges of rock" is substituted in the definition of rock excavation. Even though a contractor may choose to rip some materials, this operation may still be called rock excavation by some agencies.

The AASHTO [17] uses the term unsuitable excavation to refer to the removal and disposal of deposits of mixtures of soil and organic matter not suitable for embankment material.

If there is not sufficient material within the cross section of construction for completion of the grade, additional material may be obtained from borrow pits. This is called borrow excavation.

Common excavation consists of all excavation not otherwise classified. All excavation is paid for on a cubic meter or cubic yard basis, measured in place before excavation occurs.

14-20. CONSTRUCTION OF EMBANKMENTS

Embankments are used in construction when it is required to maintain a grade for the roadway, runway, or railway. Usually the grade is built up in a fill or embankment section from material in a cut or excavation section and is termed a rolled-earth embankment. The excavated material may be obtained from within the construction limits or from borrow pits.

Rolled-earth embankments are constructed in relatively thin layers of loose soil. Each layer is rolled to a satisfactory degree of density before the next layer is placed and the fill or embankment thus is built up to the desired height by the formation of successful layers or lifts. At the present time most agencies require layers to be from 150 to 300 mm (6 to 12 in.) thick before compaction begins, when normal soils are encountered. Specifications may permit an increase in layer thickness where large rocks are used in the lower portion of a fill, up to a maximum thickness of 600 mm (24 in.).

The layers are required to be formed by spreading the material to uniform thickness before compaction is permitted. End dumping from trucks without spreading definitely is not permitted. The only exception to this rule may be when the embankment foundation is such that it cannot support the weight of the spreading and the compacting equipment. In such cases end dumping may be permitted until sufficient thickness can support the equipment.

A close relationship between the compaction procedure and the type of soils used for embankment purposes has to be evaluated carefully. The treatment of the various types of soils is not considered in this text.

14-21. CONTROL OF COMPACTION

Practically all soils exhibit a similar relationship between moisture content and density (dry unit weight) when subjected to dynamic compaction. Practically every soil has an optimum moisture content at which the soil attains maximum density under a given compactive effort. In the laboratory this relationship usually is performed under the Standard Proctor or the Standard AASHTO Method (T99). Briefly stated, this procedure uses the soil that passes the No. 4 sieve that is placed in a 102-mm-(4-in.) diameter mold having a volume of 944 cm^3 ($\frac{1}{30}$ ft^3). The soil is placed in three layers of about equal thickness, and each layer is subjected to 25 blows from a hammer weighing 2.5 kg (5.5 lb), having a striking face 51 mm (2 in.) in diameter and falling through a distance of 305 mm (12 in).

Due to the use of heavier compaction equipment in recent years and in order to correlate more effectively laboratory procedures with field conditions, the procedure was modified and is now known as the Modified Proctor or Modified AASHTO (T180) compaction. Under the modified procedure the same mold is employed using 25 blows from a 4.5-kg (10-lb) hammer dropping a distance of 457 mm (18 in.) on five equal layers.

Regardless of which method is used, the optimum moisture and maximum density are usually found in the laboratory by a series of determinations and the results are plotted. Figure 14-11 shows the moisture–density relationship for a typical soil under dynamic

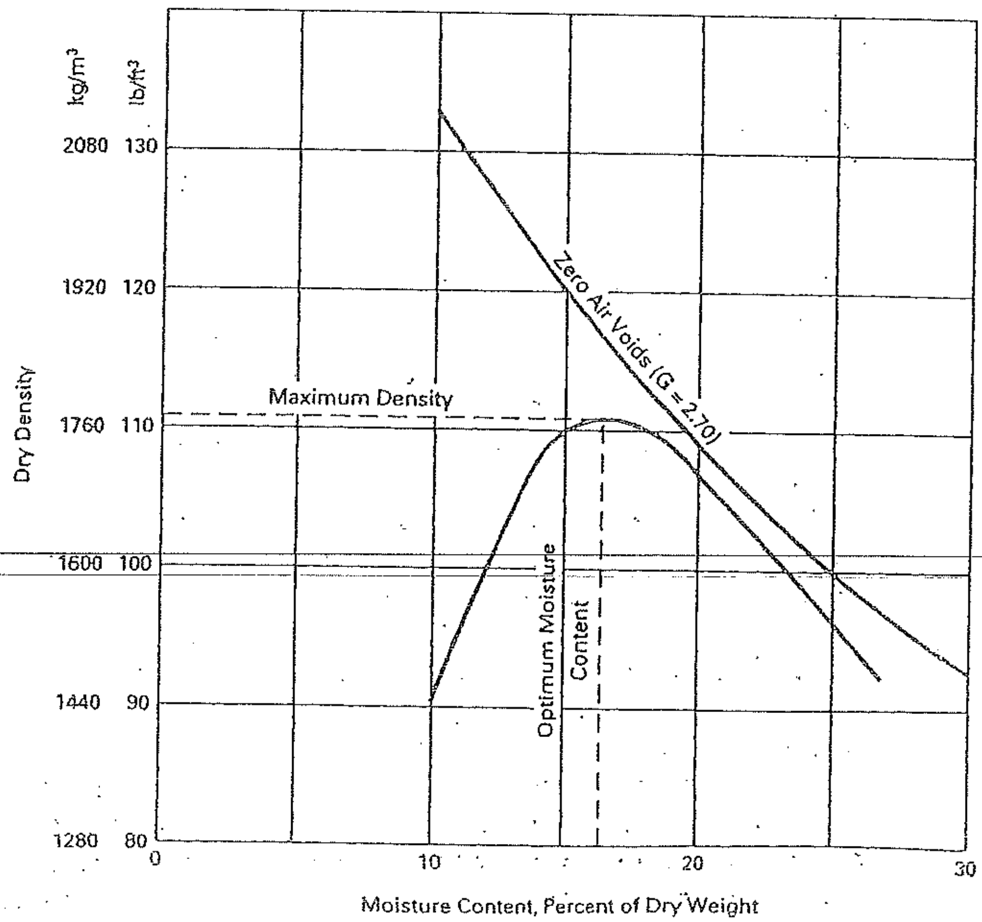


Figure 14-11 Moisture–density relationships for a typical soil under dynamic compaction. (Source: Paul H. Wright, *Highway Engineering*, Sixth Edition, John Wiley & Sons, 1996.)

compaction. The zero-air-void curve shown in Fig. 14-11 represents the theoretical density that this soil would attain at each moisture content if all the void spaces were filled with water, namely, if the soil were saturated completely.

After the laboratory density has been determined, recommendations are made for field conditions. A large majority of agencies require compaction to a certain percentage of the maximum density, as determined in the laboratory. This may range from 90 to 100 percent.

In order to determine if compaction meets laboratory requirements, field testing is required. Three methods have been used widely, which are designated as the sand, the balloon, and the heavy oil methods. Nuclear devices to measure in-place density and moisture content are being used increasingly. The details of these tests will not be discussed here.

14-22. SPECIAL EMBANKMENT FOUNDATIONS

In swampy areas, particularly where peat and other highly organic soils are encountered, special treatment may be required in the construction of embankments to prevent failure of the embankment. These methods in general use may be classified as (1) gravity subsidence, (2) partial or total excavation, (3) blasting, (4) jettying, (5) vertical sand drains, and (6) reinforcement with engineering fabrics.

In gravity subsidence a fill simply may be placed on the surface of an unsatisfactory foundation soil and allowed to settle at will, with no special treatment of the underlying soil. A temporary surface may be placed and, as settlement occurs, the temporary surface may be replaced.

Under the partial or total excavation, the undesirable soil may be removed partially or completely and backfilled with suitable material. Total excavation of the undesirable material is expensive but has its advantages in that the pavement surface may be placed immediately. Under partial excavation, gravity subsidence takes place and the temporary surface has to be replaced.

When blasting is used in swampy areas, a fill is usually placed first to a level consistently above the final grade. When the fill is complete, dynamite charges are used and so timed that the swampy material is thrust sideways and the fill material settles in place.

In jettying, the fill is made in a similar way as for blasting. The process of jettying involves the pumping of water into the underlying soil in order to liquefy it and thus aid in its displacement by the weight of the fill or embankment.

Vertical sand drains have been used increasingly in recent years. Vertical sand drains generally consist of circular holes or shafts from 450 to 600 mm (18 to 24 in.) in diameter, which are spaced from 2 to 6 m (6 to 20 ft) apart on centers beneath the embankment section and are carried beneath the layer of compressible soil. The holes are backfilled with suitable granular material. A sand blanket from 1 to 1.5 m (3 to 5 ft) thick is placed on top of the drains of the embankment. The embankment is then constructed by normal methods on top of the sand blanket. The weight of the embankment forces the water out of the sand drains and consolidation takes place.

Recently, engineers have found that engineering fabrics can be used to advantage in the construction of low fills over swampy or marshy areas. These products are permeable textile cloths or mats made from a variety of artificial fibers. Typically, the engineering fabric is placed on the weak foundation and overlain with the embankment fill. This increases the bearing capacity of the foundation and allows a higher fill to be constructed on it [8].

14-23. COMPUTING EARTHWORK QUANTITIES

The amount of earthwork on a project is usually one of the most important features in its design. Earthwork includes the excavation of material and any hauling and compaction required for completing the embankment. Payment for earthwork is based on excavated quantities only and generally includes the costs of hauling and compaction. However, an additional item of payment called *overhaul* may be used to provide for hauling of the excavated material beyond a specified *freehaul distance*.

In order to determine earth excavation and embankment requirements before construction, the gradeline for the proposed highway or railway will have to be determined and cross sections will have to be made of the original ground. Earthwork quantities then are determined by placing a section outline of the proposed roadway or railroad. These proposed outlines are called template sections. The areas in cut and the areas in fill are determined and the volumes between the sections are computed. Figure 14-12 shows template sections and original ground in cut and fill. The terms cut and fill are used for areas of the section and the terms excavation and embankment generally refer to volumes.

Cross sections are plotted on standard cross-sectional paper to any convenient scale. With the traditional U.S. system of units, a scale of 1 in. equals 5 ft vertically and horizontally is commonly used. A metric scale of 1:100 is recommended. Each cross section should show the location or station of the original ground section and template section, the elevation of the proposed grade at that station, and the areas of cut and fill for each section. The computed volumes of excavation and embankment may be placed on the sheet between two successive cross sections to facilitate the tabulation of earthwork quantities.

The areas of cut and fill may be measured by the use of a planimeter, a computation method using coordinates, or some other suitable method.

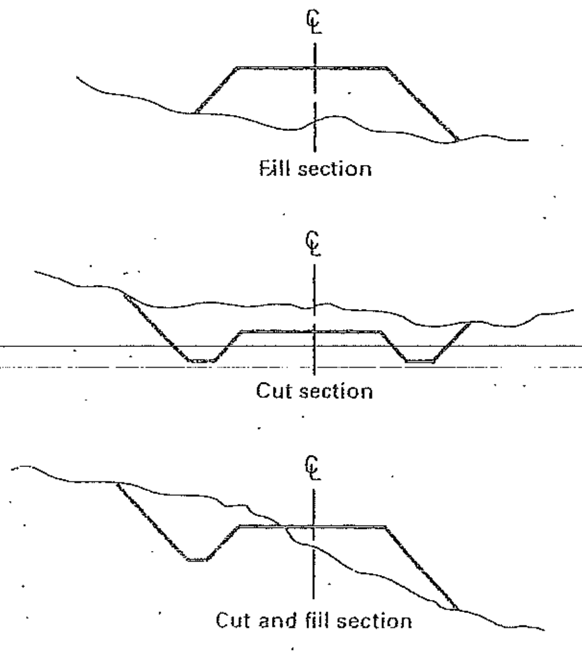


Figure 14-12 Original ground line and template sections. (Source: Paul H. Wright, *Highway Engineering*, Sixth Edition, John Wiley & Sons, 1996.)

Volumes may be computed by the average end-area method or by the prismoidal formula. The average end-area method is based on right prisms and the computed volumes are slightly in excess of those computed by the prismoidal formula. This error is small when the sections do not change rapidly; however, when sharp curves are used, prismoidal correction should be applied. The average end-area method is generally used by a majority of agencies. The formula for the average end-area method is as follows:

$$V = \tfrac{1}{2} L (A_1 + A_2) \tag{14-13}$$

where

V = volume, m³
$A_1 + A_2$ = area of end sections, m²
L = distance between end sections, m

In traditional U.S. units, this equation becomes

$$V = \frac{\tfrac{1}{2} L (A_1 + A_2)}{27} \tag{14-14}$$

where

V = volume, yd³
$A_1 + A_2$ = area of end sections, ft²
L = distance between end sections, ft

When a section changes from a cut section to a fill section, a point is reached where zero cut and zero fill occur. At this point, it will be necessary to take additional cross sections so that the proper volumes may be computed.

Manual methods described above are being supplanted rapidly by methods based on electronic computers. In typical cases and for earthwork quantities at the design stage, the computer input includes the centerline data, the cross-sectional data, template data, and other pertinent information. The computer produces a tabulation of the cut and fill volumes at each station and the difference between them adjusted for shrinkage and swell, the cumulative volumes of cut and fill, mass diagram coordinates, and slope stake coordinates. Figure 14-13 shows such a tabulation.

14-24. SHRINKAGE AND SWELL

When the freshly excavated material is hauled to an embankment, the material increases in volume; however, during the construction process of compacting the embankment, the volume decreases below that of its original volume. This is known as shrinkage. In estimating earthwork quantities, this factor must be taken into consideration. The amount of shrinkage varies with the type of soil, the depth of the fill, and the amount of compactive effort. An allowance of 10 to 15 percent frequently is made for high fills with from 20 to 25 percent for shallow fills. The shrinkage may be as high as 40 to 50 percent for some soils. This generally also allows for shrinkage due to loss of material during the hauling process and loss of material at the toe of the slope.

When rock is excavated and placed in the embankment, the material will occupy a large volume. This increase is called swell and may amount to 30 percent or more. The amount of swell is not important when small amounts of loose rock or boulders are placed in the embankment.

Roadway Design System
Identification Dallas — Earthwork Quantities Calculation Process

Earthwork Quantities List for Roadways A B

Baseline Station Number	Prism Shrink/ Swell Factor	Station Cut (SQ-M)	Station Cut (CU-M)	Adjusted Station Cut (CU-M)	Station Fill (SQ-M)	Station Fill (CU-M)	Mass Ordinate (CU-M)
6 + 700.000	0.8500	0.00	0	0	1005.44	13630	−29423
6 + 720.000	0.8500	0.00	0	0	882.76	18883	−48306
6 + 740.000	0.8500	0.00	0	0	837.13	17199	−65505
6 + 760.000	0.8500	0.00	0	0	641.98	14792	−80297
6 + 780.000	0.8500	0.00	0	0	602.44	12444	−92741
6 + 800.000	0.8500	0.00	0	0	709.73	13121	−105862
6 + 820.000	0.8500	0.00	0	0	767.68	14774	−120636
6 + 840.000	0.8500	0.00	0	0	895.94	16636	−137272
6 + 860.000	0.8500	0.00	0	0	913.35	18093	−155365
6 + 880.000	0.8500	0.00	0	0	725.23	16386	−171751
6 + 900.000	0.8500	13.30	133	113	387.25	11125	−182768
6 + 920.000	0.8500	0.21	135	115	199.34	5868	−188516
6 + 940.000	0.8500	74.38	746	634	39.54	2389	−190271
6 + 960.000	0.8500	145.71	2210	1878	14.17	537	−188930
6 + 980.000	0.8500	156.49	3022	2568	0.00	141	−186503
7 + 000.000	0.8500	204.39	3609	3067	0.00	0	−183436
7 + 020.000	0.8500	289.52	4939	4199	0.00	0	−179237
7 + 040.000	0.8500	246.17	5357	4554	0.00	0	−174683
7 + 060.000	0.8500	303.54	5497	4672	0.00	0	−170011
7 + 080.000	0.8500	252.46	5560	4726	0.00	0	−165285
7 + 100.000	0.8500	100.12	3526	2997	0.00	0	−162288
7 + 120.000	0.8500	142.87	2430	2066	0.00	0	−160222

Figure 14-13. Mass diagram data.

14-25. THE MASS DIAGRAM

A mass diagram is a graphical representation of the amount of earth excavation and embankment involved on a project and the manner in which earth is to be moved. It shows the location of balance points, the direction of haul, and the amount of earth taken from or hauled to any location. It is a valuable aid in the supervision of grading operations and is helpful in determining the amount of overhaul and the most economical distribution of material.

Figure 14-14 is a partially completed mass diagram generated from the data tabulated in Figure 14-13. Columns 5 and 8 in the figure show the cut volumes and full volumes, respectively, that have been adjusted for shrinkage. The cut and full volumes are summed algebraically to the cumulative shown for the previous station. The resulting mass ordinates are plotted for each station, as shown in Fig. 14-14.

Overhaul. The overhaul distance may be defined as the length of haul beyond a certain distance known as freehaul. This freehaul distance may be as low as 150 m (500 ft) and as long as 900 m (3000 ft) or more. Some agencies do not consider any overhaul, which

6+600.000	0
6+620.000	-5009
6+640.000	-11465
6+660.000	-12567
6+680.000	-15793
6+700.000	-29423
6+720.000	-48306
6+740.000	-65505
6+760.000	-80297
6+780.000	-92741
6+800.000	-105862
6+820.000	-120636
6+840.000	-137272
6+860.000	-155365
6+880.000	-171751
6+900.000	-182768
6+920.000	-188516
6+940.000	-190271
6+960.000	-188930
6+980.000	-186503
7+000.000	-183436
7+020.000	-179237
7+040.000	-174683
7+060.000	-170011
7+080.000	-165285
7+100.000	-162288
7+120.000	-160222
7+140.000	-158281
7+160.000	-158847
7+180.000	-161916
7+200.000	-164212
7+220.000	-163334
7+240.000	-163256
7+260.000	-164803
7+280.000	-166768
7+300.000	-165536
7+320.000	-161577
7+340.000	-158822
7+360.000	-158216
7+380.000	-153698
7+400.000	-143304
7+420.000	-126359
7+440.000	-103168
7+460.000	-78347
7+480.000	-55706
7+500.000	[illegible]
7+520.000	-31452
7+540.000	-29000
7+560.000	-32765
7+580.000	-42112
7+600.000	-52159
7+620.000	-61517
7+640.000	-69436
7+660.000	77434

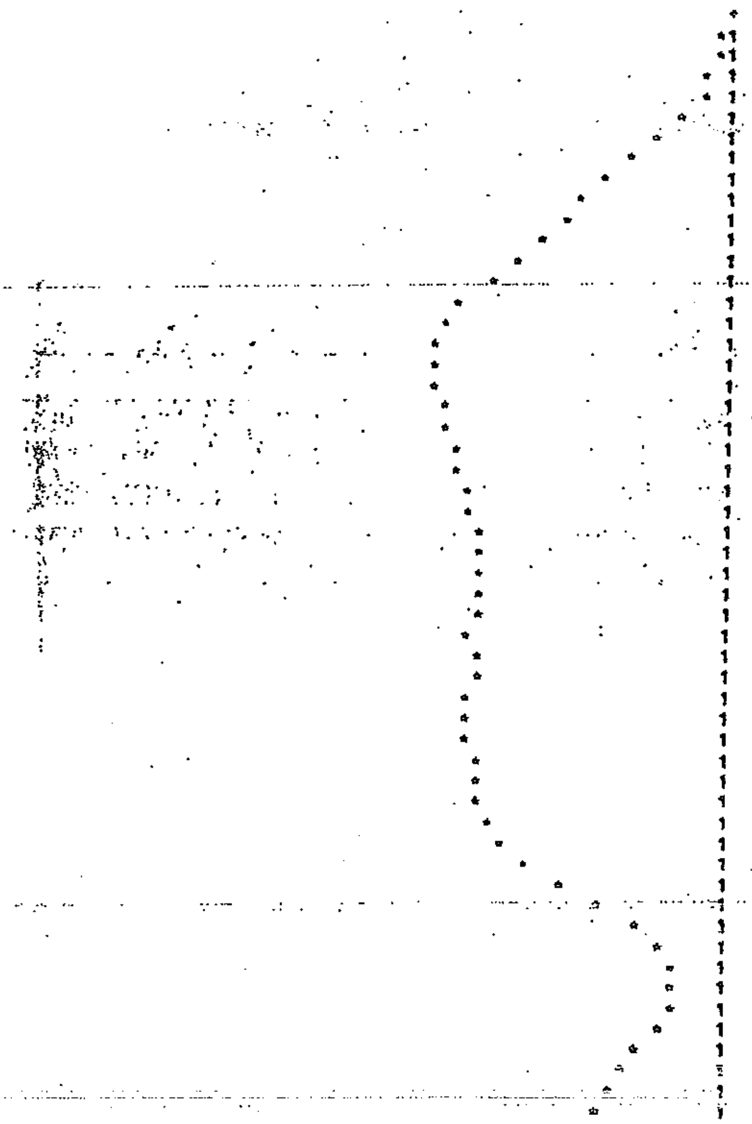


Figure 14-14 A portion of a computer-generated mass diagram.

means that the cost of excavation and hauling of the material is included in the cost of excavation. The overhaul distance is found from the mass diagram by determining the distance from the center of mass of the excavated material to the center of mass of the embankment. Under the conventional U.S. system of units, this distance is usually measured in 100-ft stations. Thus, a cubic-yard station is the hauling of 1 yd^3 of excavation one station beyond the freehaul distance. In metric units, the overhaul is the product of the amount of excavation hauled (in cubic meters) and the overhaul distance (in meters). The product may be expressed in meters to the fourth power or some compound unit such as cubic-meter-hectometer or cubic-meter-kilometer.

Several methods for determining overhaul are in use. The graphical method, the method of movements, and the planimeter method are a few. Various computer programs have been developed to perform these calculations, which result in a large saving of time. The graphical method will be illustrated here to give some idea as to the approach to the problem and theory involved. For a more detailed explanation of the other methods, reference is made to Chapter 13 of *Highway Engineering* by Wright [7].

Graphical Method of Determining Overhaul. Consider the example mass diagram shown in Fig. 14-15.

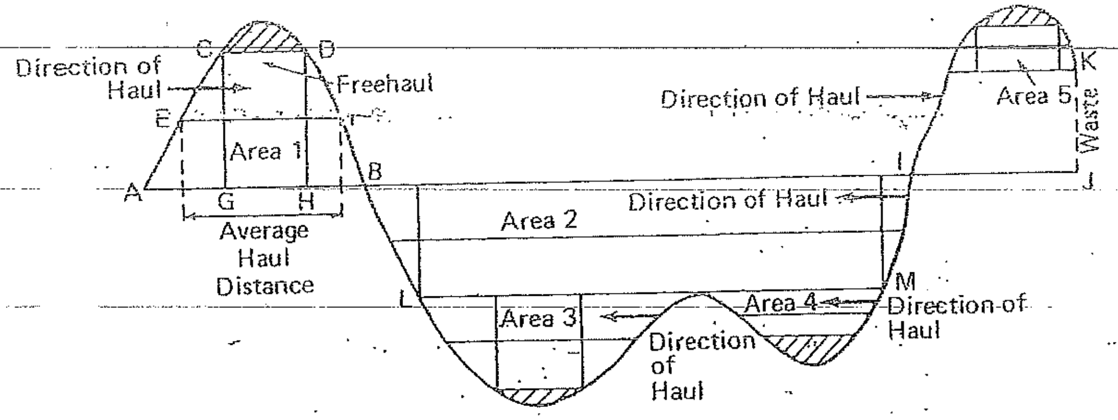


Figure 14-15 A mass diagram.

The balance points and direction of haul are as indicated. To determine the overhaul for each balance, a line equal to the freehaul distance, 1000 ft in this case, is drawn parallel to the baseline *AJ*. The shaded area indicates freehaul and is eliminated from further consideration. The next step is to drop perpendiculars from *C* and *D* to the baseline. These perpendiculars are bisected and extended to intersect the mass diagram curve at *E* and *F*. The distance *EF* is the average haul distance. The vertical distance *CG* or *DH* indicates the number of cubic yards. To compute the overhaul for this balance, it will be the average distance *EF* in stations less the freehaul distance in stations multiplied by the volume in cubic yards. This is done for each balance. Where the curve changes within a balance, the area has to be divided as indicated by the line *LM*, and the process is repeated. The summation of each area will give the total overhaul for the project. The line *KJ* indicates that so many cubic yards of material has to be wasted. If this were below the baseline *AJ*, it would indicate that borrow would have to be made.

This method gives the average haul and it has its limitations. If the slopes of the curve are more or less uniform, a minimum of error occurs. If the slopes are irregular, the method of movements or some other method should be used.

Length of Economical Haul. Where it is necessary to haul material long distances, it is sometimes more economical to waste material excavated from the roadway or railway section and borrow material from a borrow pit from within the freehaul distance. The length of economical haul can be determined by equating the cost of the excavation plus the cost of overhaul to the cost of excavation in the road or railway plus the cost of excavation from the borrow pit.

If h equals the length of haul in stations beyond the freehaul distance, e equals the cost of excavation, and o equals the cost of overhaul, then to move 1 yd^3 of material from cut to fill, the cost will be $e + ho$, and the cost to excavate from cut, waste the material, borrow, and place 1 yd^3 in the fill will equal $2e$. Assuming that the cost of the roadway excavation is equal to the cost of borrow excavation,

$$e + ho = 2e$$

and

$$h = \frac{e}{o} \text{ stations}$$

In metric units, if e is expressed in dollars per cubic meter and o in dollars per cubic meter-kilometer, h would be expressed in kilometers.

DESIGN OF PAVEMENTS

Generally speaking, pavements (and bases) may be divided into two broad classifications or types: flexible and rigid. A flexible pavement structure maintains intimate contact with and distributes loads to the subgrade and depends on aggregate interlock, particle friction, and cohesion for stability [19]. Thus, the classical flexible pavement includes primarily those pavements that are composed of a series of granular layers topped by a relatively thin high-quality bituminous wearing surface. As commonly used in the United States, the term "rigid pavement" is applied to wearing surfaces constructed of Portland cement concrete. A pavement constructed of concrete is assumed to possess considerable flexural strength that will permit it to act as a beam and allow it to bridge over minor irregularities, which may occur in the base or subgrade on which it rests.

14-26. ELEMENTS OF A FLEXIBLE PAVEMENT

The principal elements of a flexible pavement include a wearing surface, base, subbase (not always used), and subgrade. (See Fig. 14-16.) The wearing surface may range in thickness from less than 25 mm (1 in.) in the case of a bituminous surface treatment used for low-cost, light-traffic roads to 150 mm (6 in.) or more of asphalt concrete used for heavily traveled routes. The wearing surface must be capable of withstanding the wear and abrasive effects of moving vehicles and must possess sufficient stability to prevent it from shoving and rutting under traffic loads. In addition, it serves a useful purpose in preventing the entrance of excessive water into the base and subgrade.

The base is a layer (or layers) of natural or processed material, such as graded crushed rock, gravel, asphalt concrete, or cement treated materials. Its principal purpose is to distribute or spread the stresses created by wheel loads acting on the wearing surface so that the stresses transmitted to the subgrade will not cause excessive deformation or displace-

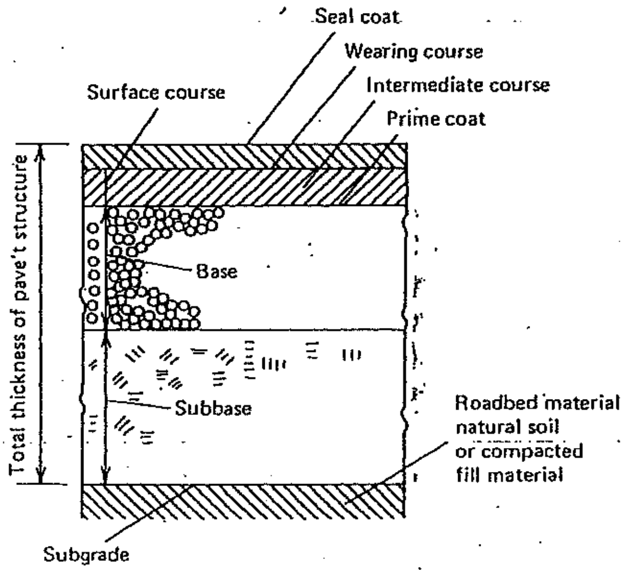


Figure 14-16 Typical flexible pavement cross section. (Courtesy Transportation Research Board.)

ment of that foundation layer. The base must also be resistant to damage by capillary water and/or frost action. Locally available materials are extensively used for base construction, and materials preferred for this type of construction vary widely in different sections of the country.

A subbase of granular material or stabilized material may be used in areas where frost action is severe or in locations where the subgrade is extremely weak. In the interests of economy, it may also be used in locations where suitable subbase materials are cheaper than base materials of higher quality.

The subgrade is the foundation layer, the structure that must eventually support all of the loads that come onto the pavement. Some transportation agencies use the term "basement soil." Sometimes, this layer will simply be the natural earth surface. In other instances it will be the compacted soil existing in a cut section or the upper layer of an embankment section.

14-27. FACTORS THAT AFFECT FLEXIBLE PAVEMENT DESIGN

The principal factors that affect the thickness design of a flexible pavement are:

1. Traffic loading
2. Material characteristics
3. Climate or environment

The primary loading factors that are important in flexible pavement design are:

1. Magnitude of axle (and wheel) loads
2. Volume and composition of axle loads
3. Tire pressure and contact area

The magnitude of maximum loading is normally controlled by legal load limits. Traffic surveys and loadometer studies are often used to establish the relative magnitude and occurrence of the various loadings to which a pavement is subjected. The prediction or estimation of the total traffic that will use a pavement during its design life is a very difficult but important task. Most design procedures provide for an increase in traffic volume on the basis of experience by using some estimated growth rate.

Proper design of flexible pavement systems requires a thorough understanding of the important characteristics of the materials of which the pavement is to be composed and on which it is to be supported. The relevant material characteristics may include gradation, strength or stability, and resistance to the effects of repeated loadings. Various standard test methods are available for determining the desired properties (documented elsewhere [20, 21]).

The climate or environment in which a flexible pavement is to be established has an important influence on the behavior and performance of the various materials in the pavement and subgrade. Probably the two climatic factors of major significance are temperature and moisture.

The magnitude of temperature and its fluctuations affect the properties of certain materials. For example, high temperatures cause asphaltic concrete to lose stability whereas at low temperatures asphaltic concrete becomes very hard and stiff. Low temperature and temperature fluctuations are also associated with frost heave and freeze–thaw damage.

Moisture also has an important influence on the behavior and performance of many materials. Moisture is an important ingredient in frost-related damage. Subgrade soils and other pavement materials weaken appreciably when saturated, and certain clayey soils exhibit substantial moisture-induced volume change.

14-28. THE AASHO ROAD TEST

The single most costly element of the nation's highway system is the pavement structure [15]. Seeking to control this cost, state and federal highway agencies have been involved in a continuous program of pavement research since the late 1950s. The most significant pavement research initiative was the AASHO[3] road test, a large-scale project undertaken cooperatively by the various states, the federal government, and industry groups in 1958. In this project, special test sections of variable thicknesses were constructed in Ottawa, Illinois, and subjected to repeated loadings from traffic that included both single- and tandem-axle vehicles. Each test section was subjected to thousands of load repetitions. The research project dealt with both flexible and rigid pavements.

The AASHO road test and subsequent research led to the publication of a series of guides for the design of pavements, first in 1961, then in 1972, 1981, 1986, and 1993. The following section summarizes the design approach for flexible pavements recommended by the AASHTO Design Committee in the 1993 edition of the guide [19].

14-29. THE AASHTO DESIGN METHOD FOR FLEXIBLE PAVEMENTS

One of the products of the AASHO road test was the "pavement serviceability concept." In essence, this involves the measurement, in numerical terms, of the behavior of the pavement under traffic, that is, its ability to serve traffic at some instant during its life [22].

Such an evaluation can be made on the basis of a systematic but subjective rating of the riding surface by individuals who travel over it. Or pavement serviceability can be evaluated by means of certain measurements made on the surface, as was done on the test road. The AASHTO design method employs a *present serviceability index* (PSI) that is measured on a scale of 0 to 5, with 0 corresponding to an impossible road and 5 representing a perfect road.

The AASHTO method considers the change in the PSI rating over its performance period:

$$\Delta PSI = p_o - p_t$$

where

p_o = original or initial serviceability index
p_t = terminal or lowest allowable serviceability index

The original or initial PSI rating for flexible pavements generally falls between 4.0 and 4.5. A value of 4.2 was reported for the flexible pavements tested at the AASHO test road project.

The value of p_t is the lowest allowable PSI rating that will be tolerated before rehabilitation, resurfacing, or reconstruction is required. The AASHTO recommends p values of 2.5 or higher for the design of major highways and 2.0 for highways with low traffic volumes.

Time Constraints. With the AASHTO method, the designer must select two time-related variables or constraints: the performance period and the analysis period. The performance

[3] At the time of the test, the AASHTO was called the American Association of State Highway Officials (AASHO).

period is the period of time that an initial pavement structure will last before it needs rehabilitation. In other words, it is the time elapsed as a new, reconstructed, or rehabilitated structure deteriorates from its initial serviceability to its terminal serviceability.

The analysis period is the period of time for which the analysis is conducted. It is similar to the term "design life" used by designers in the past. It is suggested that consideration be given to extending the analysis period to include one rehabilitation, or longer in the case of high-volume urban freeways.

The approach allows the designer to consider strategies ranging from the initial structure lasting the entire analysis period to stage construction with overlays at planned intervals.

Traffic. The AASHTO design procedure is based on the cumulative expected load applications during the analysis period. The design committee handled the problem of mixed traffic by first adopting an 18,000-lb single-axle load as a standard and then developing a series of "equivalence factors" for each axle weight group. If the estimated traffic to be used in design can be broken down into axle load groupings, the number of load applications in each group can be multiplied by the equivalence factor to determine the number of 18,000-lb axle loads that would have an equivalent effect on the pavement structure. The committee tabulated equivalency factors for a range of axle loads, structural numbers, and p_t values [19].

For purposes of pavement design, an estimate must be made of the cumulative equivalent single-axle load (ESAL) applications *in the design lane.* It may be necessary, therefore, to multiply the estimated cumulative two-directional 18-kip single-axle load applications by a directional factor and, if the facility has two or more lanes in one direction, by a lane distribution factor.

Roadbed Soil. In the AASHTO design procedure, the roadbed soil strength is characterized by its modulus of elasticity, or *resilient modulus.* In the standard method of test for the resilient modulus of subgrade soils, a specially prepared and conditioned specimen is subjected to repeated applications of axial deviator stress of fixed magnitude, duration, and frequency. During the test, the specimen is subjected to a static all-around stress in a triaxial pressure chamber. The test is intended to simulate the conditions that exist in pavements subjected to moving wheel loads. Detailed procedures for the test are given by AASHTO Designation T274-82 [21].

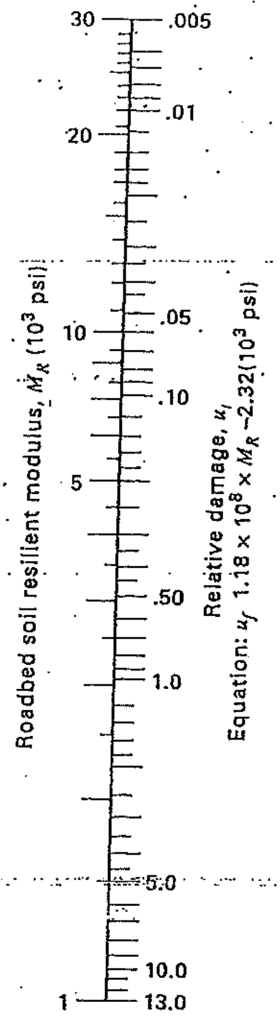
Suitable equations have also been published to make it possible to estimate the resilient modulus of a soil from other measures of subgrade strength such as the California bearing ratio (CBR) and the *R*-value.

The AASHTO [19] recommends that laboratory resilient modules tests be performed on representative samples in stress and moisture conditions simulating primary moisture seasons. The purpose of determining seasonal moduli is to account for the relative damage a pavement is subjected to during each season of the year. This is done by calculating an effective roadbed soil resilient modulus, a weighted value that gives the equivalent annual damage obtained by considering each season independently in the performance equation and summing the damage.

Figure 14-17 illustrates the estimation of an effective roadbed resilient modulus by averaging the relative damage values for 12 months of the year. The resilient modulus of the roadbed soil is determined for each month and entered in the second column of the figure. The corresponding relative damage values are determined from the vertical scale and averaged. The average relative damage value (0.31 in this example) is then used to determine the effective resilient modulus of the roadbed soil from the vertical scale (e.g., $M_r = 5000$ psi).

Month	Roadbed soil modulus, M_R (psi)	Relative damage u_f
Jan.	20,000	0.01
Feb.	20,000	0.01
Mar.	2,500	1.51
Apr.	4,000	0.51
May	4,000	0.51
June	7,000	0.13
July	7,000	0.13
Aug.	7,000	0.13
Sept.	7,000	0.13
Oct.	7,000	0.13
Nov.	4,000	0.51
Dec.	20,000	0.01
Summation: $\Sigma u_f =$		3.72

Average: $\bar{u}_f = \frac{\Sigma u_f}{n} = \frac{3.72}{12} = 0.31$



Effective roadbed soil resilient modulus, M_R (psi) = 5,000 (corresponds to $\bar{u}_f$)

Figure 14-17 Chart for estimating effective roadbed soil resilient modulus for flexible pavements using the serviceability criteria. (Source: *AASHTO Guide for Design of Pavement Structures*, copyright 1993, American Association of State Highway and Transportation Officials. Used by permission.)

Environmental Effects. Reference 19 provides a method for accounting for certain detrimental environmental effects on pavement performance. It recommends that if roadbed swelling or frost heave can lead to a significant loss of serviceability during the analysis period, the serviceability loss from these effects should be added to that resulting from the cumulative axle loadings.

Reliability. The structural design of a pavement is fraught with uncertainty. The design process is extremely complex, and its performance period will depend on the imposed traffic loadings, roadbed soil factors, climate, its structural design (e.g., layer types and thicknesses), and the intended serviceability.

The AASHTO design committee introduced the concept of reliability into the design process to incorporate some degree of certainty that the various design alternatives will last the analysis period.

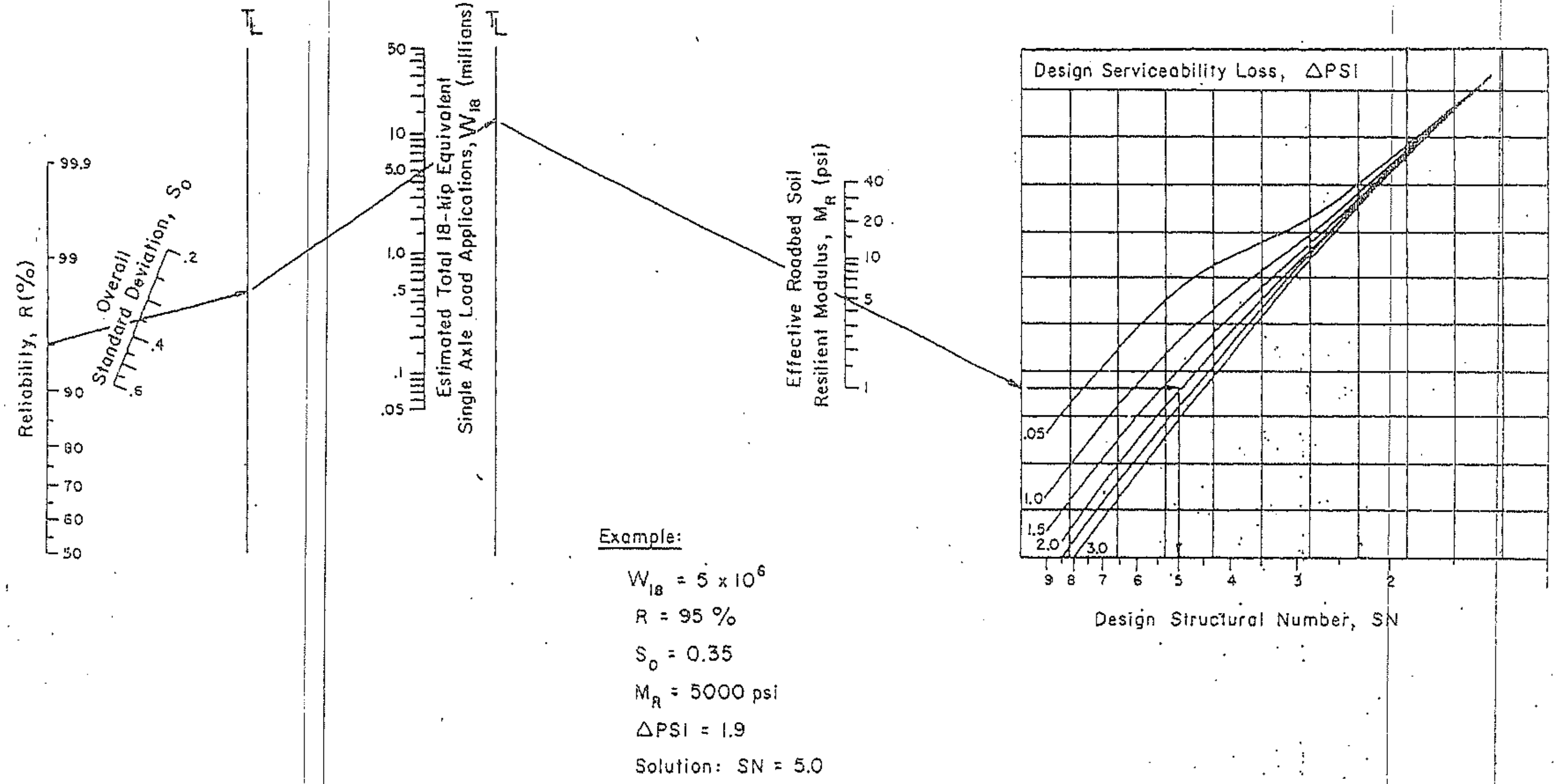


Figure 14-18 A design chart for flexible pavements that is based on using mean values for each input. (Source: *AASHTO Guide for Design of Pavement Structures*, copyright 1993, American Association of State Highway and Transportation Officials. Used by permission.)

The reliability concept allows the designer to choose a predetermined level of assurance that the pavement will serve the period for which it was designed. Generally speaking, as the volume of traffic, difficulty of rerouting traffic, and public expectation of using the facility increase, the risk of failing to perform to expectations must be minimized. This may be accomplished by selecting higher levels of reliability. Recommended levels of reliability vary from 50 to 80 percent for local roads to 80 to 99.9 percent for Interstate facilities.

In addition to a level of risk, the reliability concept takes into account the chance variation in traffic prediction and the normal variation in pavement performance prediction. This is accomplished by the selection of an overall standard deviation. The results of the AASHO road test and subsequent experience indicate that the overall standard deviation for flexible pavements is about 0.45 years.

Design Chart. The AASHTO design chart for flexible pavements is shown in Fig. 14-18. This nomograph provides a *design structural number,* SN, required for specific conditions of reliability, overall standard deviation, estimated future traffic, effective resilient modulus of the roadbed material, and design serviceability loss.

The design structural number is an index number that may then be converted to thickness of flexible pavement layers through the use of suitable layer coefficients related to the types of material being used. This is expressed mathematically as

$$SN = a_1 D_1 + a_2 D_2 + a_3 D_3 \tag{14-15}$$

where

a_1, a_2, a_3 = layer coefficients representative of the surface, base, and subbase courses, respectively

D_1, D_2, D_3 = actual thicknesses (in inches) of the surface, base, and subbase courses, respectively

In the AASHO road test, the layer coefficients were 0.44, 0.14, and 0.11 for asphaltic concrete, crushed limestone, and sand-gravel, respectively. Reference 19 gives specific guidance in the choice of layer coefficients based on laboratory test of layer materials.

The AASHTO guide [19] provides a method for accounting for the effects of untreated base and subbase drainage on pavement performance. The method involves incorporating modifying coefficients m_2 and m_3 into the structural number equation to allow for the drainage effects:

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3 \tag{14-16}$$

Recommended values for m_2 and m_3 are given in the guide [19].

The structural number equation does not have a single solution. Many combinations of layer thicknesses and types will give satisfactory designs, and it is the designer's responsibility to determine the most appropriate design based on availability of materials, economy, and other factors.

When selecting appropriate values for the layer thicknesses, the designer should adhere to the suggested minimum practical thicknesses of the two upper layers, as shown in Table 14-4.

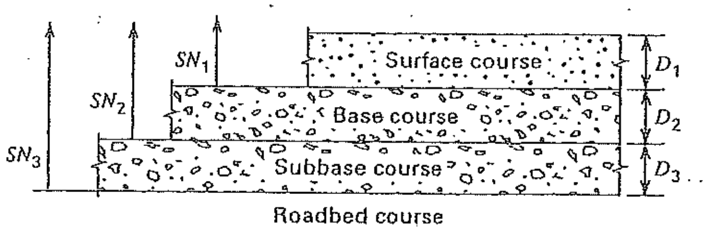
Layered-Design Analysis. By considering the pavement structure as a layered system, it is possible to compute the minimum thickness of the various layers. First, from Fig.

Table 14-4 Minimum Practical Thicknesses for Each Pavement Course

Traffic, ESAL's	Asphalt Concrete (in.)	Aggregate Base (in.)
Less than 50,000	1.0 (or surface treatment)	4
50,000–150,000	2.0	4
150,000–500,000	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

Source: AASHTO Guide for Design of Pavement Structures, copyright 1993 by the American Association of State Highway and Transportation Officials, Washington, DC, 1993. Used by permission.

14-18, using the resilient modulus of the roadbed soil, the structural number required over the roadbed soil is determined. Similarly, the structural number over the subbase and the base layers are determined from Fig. 14-18 using the applicable strength values for each layer. By working with the differences between the structural numbers required over each layer, the minimum allowable thickness of any given layer can be computed using the equations shown in Fig. 14-19.



$$D^*_1 \geq \frac{SN_1}{a_1}$$

$$SN^*_1 = a_1 D^*_1 \geq SN_1$$

$$D^*_2 \geq \frac{SN_2 - SN_1}{a_2 m_2}$$

$$SN^*_1 + SN^*_2 \geq SN_2$$

$$D^*_3 \geq \frac{SN_3 - (SN^*_1 + SN^*_2)}{a_3 m_3}$$

1) a, D, m and SN are as defined in the text and are minimum required values.

2) An asterisk with D or SN indicates that it represents the value actually used, which must be equal to or greater than the required value.

Figure 14-19 Procedure for determining thicknesses of layers using a layered analysis approach. (Source: *AASHTO Guide for Design of Pavement Structures,* copyright 1993, American Association of State Highway and Transportation Officials. Used by permission.)

EXAMPLE 14-2

Consider the example illustrated in Fig. 14-18. For the conditions shown, the required structural number SN = 5.0. Suppose we wish to design a flexible pavement with a wearing surface, base, and subbase with layer coefficients and resilient modulus values as shown:

Wearing surface	$a_1 = 0.40$	$E_1 = 360{,}000$ psi
Base	$a_2 = 0.12$	$E_2 = 25{,}000$ psi
Subbase	$a_3 = 0.09$	$E_3 = 12{,}000$ psi

Assume that the base and subbase are well drained and the structural layer coefficients will not need to be modified (i.e., $m_2 = 1.0$, $m_3 = 1.0$).

By Eq. 14-15,

$$5.0 = 0.40D_1 = 0.12D_2 + 0.09D_3$$

From 14-18, using E_2 as M_r, the required structural number for the base course is $SN_1 = 3.4$. The depth of the surface course $D_1 > 8.5$ in.

From Fig. 14-18, using E_3 as M_r, the structural number for the subbase course is $SN_2 = 3.8$. The depth of the base course $D_2 > [SN_2 - a_1D_1]\ /a_2 = [3.8 - 0.4(8.5)]$ $/0.12 = 3.3$ in., but from Table 14-4, we use a minimum depth of 6 in.

The depth of the subbase $D_3 > [SN_3 - (a_1D_1 + a_2D_2)]/a_3 = [5.0 - 0.4\ \ (8.5) + 0.12(6)]/0.09\ D_3 > 9.7$ in., say 10 in.

Checking the solution using Eq. 14-15 yields SN = 0.4(8.5) + 0.12(6) + 0.09(10) = 5.02 > 5.0. The solution checks.

14-30. ELEMENTS OF A RIGID PAVEMENT

Rigid pavements consist of a Portland cement concrete slab placed on a uniform subbase or subgrade. Such pavements commonly are one of four types: (1) plain pavements, (2) plain-doweled pavements, (3) reinforced pavements, or (4) continuously reinforced pavements.

As Portland cement concrete cures, there is a tendency for the slab to crack. The designer must recognize this tendency and provide for load transfer between adjacent slabs. Four approaches have been used to provide for load transfer. In plain pavements, closely spaced joints with typical spacing of about 4.5 m (15 ft) are provided to control cracking, and load transfer is obtained by aggregate interlock between the cracked faces of the joint. In plain-doweled pavements, relatively close joint spacings commonly not more than 6 m (20 ft) are similarly used to control cracking, but smooth steel dowel bars are installed at each joint to ensure proper load transfer. Reinforced concrete pavements contain reinforcing steel mats as well as dowel bars and are built with joints spaced up to 12 m (40 ft). Some agencies have constructed continuously reinforced concrete pavements without contraction joints. These pavements tend to develop minute cracks at close intervals and provide load transfer by aggregate interlock at the crack faces held together by steel reinforcement.

Portland cement concrete pavements may be placed directly on a carefully prepared subgrade but more commonly are constructed on a relatively thin subbase of sand, soil-cement, or some other subbase.

Typical sections of concrete pavements are shown in Fig. 14-20.

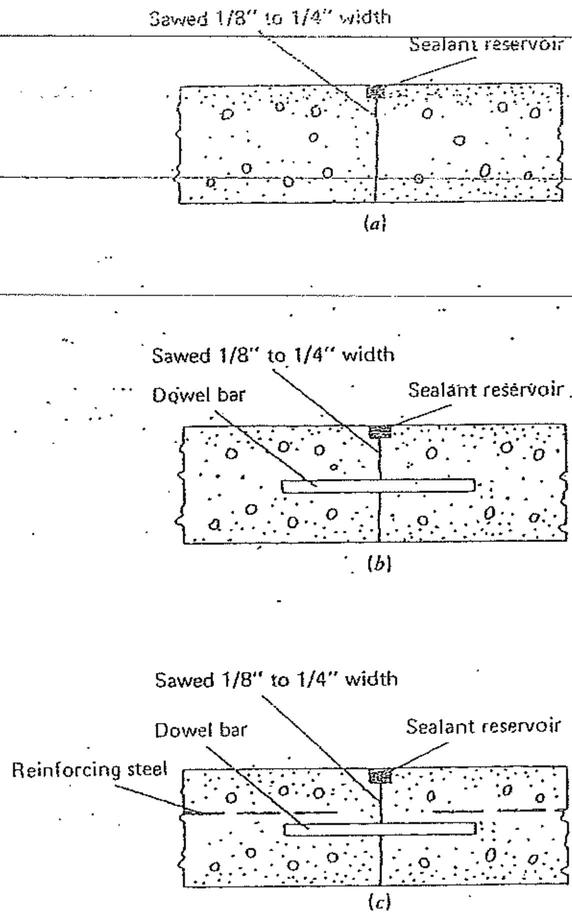


Figure 14-20 Types of concrete pavements. (*a*) Plain pavement. (*b*) Plain-doweled pavement. (*c*) Reinforced pavement.

14-31. FACTORS THAT AFFECT RIGID-PAVEMENT DESIGN

Like flexible pavements, the thickness of rigid pavements that is required for satisfactory performance depends on traffic loading, material characteristics, and environmental or climatic factors. The numbers and weights of heavy axle loads that are expected during the pavement's service life are major factors in thickness design. Reliable estimates are needed not only of the volume of traffic expected during the design life of the pavement but also of the distribution of the traffic by various axle load categories. These estimates are normally based on data routinely collected at the highway agency's loadometer weight stations or on special traffic studies on routes with a distribution of traffic similar to the project being designed. When specific axle load data are not available, composite axle load distributions that represent different categories of road and street types can be used.

The strength of the concrete is the most important material characteristic that affects the design of the pavement. Since direct compressive stresses in concrete pavements due to wheel loads are very small in relation to the compressive strength of the material, they can be ignored. Flexural strength is a key factor in thickness design, since the flexural stress produced by a heavy wheel load often is more than one-half the flexural strength.

Flexural strength is measured by modulus of rupture tests on beams subjected to third-point loadings. Typically, 28-day to 90-day strengths are used for roads and streets, since very few stress repetitions occur during the first 90 days of pavement life, compared with the millions of repetitions that occur after that time. Also, the strength of concrete increases with age.

Another major factor in thickness design is subgrade or subbase support, which may be measured by the Westergaard modulus of subgrade reaction k. To determine k, a plate-bearing test is performed, generally using a 762-mm (30-in.) plate. The k-value is equal to the load in pounds per square inch divided by the deflection in inches caused by that load. Its units are therefore psi per inch or pounds per cubic inch. In the metric system, the units are kilopasacals per millimeter. Because plate-bearing tests are expensive and time consuming, the k-value is sometimes estimated by its correlation to simpler tests, such as the CBR or soil classification.

The k-value can be increased by using a treated or untreated subbase, although the use of a subbase for this purpose alone seldom would be economical.

The most troublesome environmental factor that can affect concrete pavement performance is the existence of a surplus of moisture in the subgrade soil. This condition under a pavement that is subjected to frequent heavy wheel loadings can cause a problem called "pumping." This phenomenon may be defined as follows. The movement of slab ends under traffic loads causes the extrusion, or "pumping," of a portion of the subgrade material at joints, in cracks, and along the edges of the pavement. The amount of soil removed by pumping may be sufficient to cause a sizable reduction in subgrade support for the slab and may result in eventual failure of the pavement. Where pavements are to be constructed on a fine-grained subgrade soil, a granular subbase 3 to 6 in. in thickness should be used. In areas where pumping may be a problem, it is desirable to minimize or eliminate expansion joints and ensure that contraction joints are adequately sealed to prevent the intrusion of surface water.

14-32. THE PORTLAND CEMENT ASSOCIATION THICKNESS DESIGN METHOD

Here we present the essentials of one method of design of rigid pavements, in the belief that this method is typical of modern approaches to this subject. The method presented is that contained in *Thickness Design for Concrete Pavements*, published by the Portland Cement Association (PCA) [23].

According to PCA engineers, design considerations that are vital to the satisfactory performance and long life of a concrete pavement are reasonably uniform support for the pavement, elimination of pumping by the use of a thin treated or untreated base course, adequate joint design, and a thickness that will keep load stresses within safe limits. The overall objective of the design procedure is to determine the minimum thickness that will give the least annual cost.

The PCA recommends two design criteria:

1. Fatigue, to keep pavement stresses from repeated loads within acceptable limits to prevent fatigue cracking
2. Erosion, to limit the effects of pavement deflections at joints, corners, and the edges of slabs in order to control the erosion of foundation materials

Concrete is subject to fatigue, as are other construction materials. In the PCA design concept, a fatigue failure occurs when a material ruptures under continued repetitions of loads that cause stress ratios of less than 1. The stress ratio is the ratio of flexural stress to the modulus of rupture.

Flexural fatigue research on concrete has shown that the number of stress repetitions to failure increases as the stress ratio decreases; studies show that, if the stress ratio is less than 0.55, concrete will withstand virtually unlimited stress repetitions without loss in load-bearing capacity. To be conservative, designers reduced this ratio to 0.50.

In the past, thickness design methods for concrete pavements included an allowance for impact of moving loads. The PCA method discards this concept, substituting "load safety factors" in the design. The following load safety factors are recommended: Interstate and other multiple-lane projects where there will be an uninterrupted flow of heavy truck traffic, 1.2; highways and streets with moderate volumes of truck traffic, 1.1; and streets and highways that carry small volumes of truck traffic, 1.0.

Comprehensive analyses of concrete stresses and deflections at pavement joints, corners, and edges have established that the most critical axle load positions are:

1. For stresses, when the truck wheels are placed at or near the pavement edge and midway between the joints
2. For pavement deflections, at the slab corner when an axle load is placed at the joint with the wheels at the corner

It is known that only a small percentage of trucks travel with their outside wheels placed at the edge. Most truck drivers drive with their outside wheels placed about 0.6 m (2 ft) from the pavement edge. The PCA design procedure is based on the assumption that 6 percent of the trucks travel with the outside tire at or beyond the pavement edge. The maximum stresses and deflections result from those loadings. At increasing distances inward from the pavement edge, the magnitudes of stresses and deflections decrease but the frequency of load applications increases.

The PCA engineers analyzed the distributions of truck placements and accounted for the variability of positioning of loadings in their design figures and tables. They computed the fatigue incrementally for various load placements inward from the edge of the slab. They expressed the results in terms of an equivalent load factor that, when multiplied by the edge-load stress, gives the same degree of fatigue consumption that would result from a given load distribution. A similar approach was used to account for the effects of load placements on deflections.

Procedures recommended by the PCA are based on a design period of 20 years.

The PCA design procedure takes into account the effects of the erosion of foundation and shoulder materials due to pavement deflections at slab edges, joints, and corners. This analysis recognizes modes of pavement distress such as pumping, faulting, and shoulder problems that are unrelated to fatigue.

The PCA engineers worked out a number of design examples, one of which is summarized below.

EXAMPLE 14-3[a]

Determine the thickness of a concrete pavement for a rural secondary road that will serve a design average daily traffic of 720. The design period is 40 years. Designers estimate that the truck traffic volume (ADTT) is 2.5 percent of the total. Thus, ADTT = 720(0.025) = 18. Truck traffic each way is 18/2 = 9. For the design period of 40 years, there will be $9 \times 365 \times 40 = 131{,}400$ trucks using each lane. Based on a composite axle load distribution for rural secondary roads (not shown here but given in reference 23), the expected repetitions of single axles and tandem axles were computed and are shown on the design worksheet, Fig. 14-21.

[a]Conventional U.S. units are used in this example in order to be consistent with available design charts.

Calculation of Pavement Thickness

Project Design 2A, two-lane secondary road

Trial thickness 6.0 in.
Subbase-subgrade k 100 pci
Modulus of rupture, MR 650 psi
Load safety factor, LSF 1.0

Doweled joints, yes ___ no ✓
Concrete shoulder yes ___ no ✓
Design period 40 years

no subbase

Axle load, kips	Multiplied by LSF 1.0	Expected repetitions	Fatigue analysis		Erosion analysis	
			Allowable repetitions	Fatigue, percent	Allowable repetitions	Damage, percent
1	2	3	4	5	6	7

8. Equivalent stress 411
9. Stress ratio factor 0.632
10. Erosion factor 3.40

Single Axles

22	22	130	340	38.2	120,000	0.1
20	20	550	2,000	27.5	210,000	0.3
18	18	2,080	13,000	16.0	380,000	0.5
16	16	5,000	80,000	6.2	740,000	0.7
14	14	7,370	800,000	0.9	1,600,000	0.5
12	12	16,290	Unlimited	0	4,200,000	0.4
10	10	26,930	"	0	15,000,000	0.2
8	8	63,500	"	0	Unlimited	0
6	6	94,120			"	0

11. Equivalent stress 348
12. Stress ratio factor 0.535
13. Erosion factor 3.52

Tandem Axles

36	36	550	190,000	0.3	160,000	0.3
32	32	9,140	2,500,000	0.3	310,000	2.9
28	28	9,000	Unlimited	0	660,000	1.4
24	24	5,150	"	0	1,700,000	0.3
20	20	7,500	"	0	5,400,000	0.1
16	16	9,860			26,000,000	0
12	12	18,300			Unlimited	0
8	8	11,250			"	
			Total	89.4	Total	7.7

Figure 14-21 Design worksheet. (Courtesy Portland Cement Association.)

The PCA engineers evaluated two designs (2A and 2B). For design 2A, the *k*-value for the subgrade was taken to be 100 lb/in^3. The load safety factor is 1.0, with a modulus of rupture of the concrete of 650 lb/in^2.

The engineers assumed a trial depth of 6.0 in. Key calculations are shown on the design worksheet.

For the fatigue analysis, equivalent stresses are determined from Table 14-5 for single and tandem axles and are entered on the design worksheets as items 8 and 11, respectively. The equivalent stresses are then divided by the concrete modulus of rupture and entered as items 9 and 12 (stress ratio factors). Next, the allowable repetitions are determined for each loading category from Figure 14-22. The percentage of fatigue is then calculated by dividing the expected repetitions (column 3) by the allowable repetitions (column 4). The sum of the percentages of fatigue used is then totaled for all loading categories, and for this example is 89.4 percent.

In a similar way, the erosion analysis involves determining the allowable repetitions to ensure that harmful erosion of the foundation and shoulder materials does not occur from pavement deflections at slab edges, joints, and corners.

To perform this analysis, erosion factors are determined from Table 14-6 and recorded as items 10 and 13 for single- and tandem-axle loading, respectively. Allowable repetitions are then determined from Figure 14-23 and are recorded in column 6 for each loading category.

The damage due to erosion is then compiled for each loading condition by dividing the expected repetitions (column 3) by the allowable repetitions (column 6). The total erosion damage (percent) is determined by summing the incremental damages (column 7). For this example, the erosion damage is 7.7 percent.

The totals of fatigue use and erosion damage use of 89.4 and 7.7 percent, respectively, show that the 6.0-in. thickness is satisfactory for the given conditions.

Table 14-5 Equivalent Stress: No Concrete Shoulder (Single Axle/Tandem Axle)

Slab Thickness (in.)	k of Subgrade-Subbase, pci						
	$k = 50$ lb/in.^{3}	$k = 100$ lb/in.^{3}	$k = 150$ lb/in.^{3}	$k = 200$ lb/in.^{3}	$k = 300$ lb/in.^{3}	$k = 500$ lb/in.^{3}	$k = 700$ lb/in.^{3}
4	825/679	726/585	671/542	634/516	584/486	523/457	484/443
4.5	699/586	616/500	571/460	540/435	498/406	448/378	417/363
5	602/516	531/436	493/399	467/376	432/349	390/321	363/307
5.5	526/461	464/387	431/353	409/331	379/305	343/278	320/264
6	465/416	411/348	382/316	362/296	336/371	304/246	285/232
6.5	417/380	367/317	341/286	324/267	300/244	273/220	256/207
7	375/349	331/290	307/262	292/244	271/222	246/199	231/186
7.5	340/323	300/268	279/241	265/224	246/203	224/181	210/169
8	311/300	274/249	255/223	242/208	225/188	205/167	192/155
8.5	285/281	252/232	234/208	222/193	206/174	188/154	177/143
9	264/264	232/218	216/195	205/181	190/163	174/144	163/133
9.5	245/248	215/205	200/183	190/170	176/153	161/134	151/124
10	228/235	200/193	186/173	177/160	164/144	150/126	141/117
10.5	213/222	187/183	174/164	165/151	153/136	140/119	132/110
11	200/211	175/174	163/155	154/143	144/129	131/113	123/104
11.5	188/201	165/165	153/148	145/136	135/122	123/107	116/98
12	177/192	155/158	144/141	137/130	127/116	116/102	109/93
12.5	168/183	147/151	136/135	129/124	120/111	109/97	103/89
13	159/176	139/144	129/129	122/119	113/106	103/93	97/85
13.5	152/168	132/138	122/123	116/114	107/102	98/89	92/81
14	144/162	125/133	116/118	110/109	102/98	93/85	88/78

Source: Thickness Design for Concrete Highway and Street Pavements, Portland Cement Association. Skokie, IL, 1984.

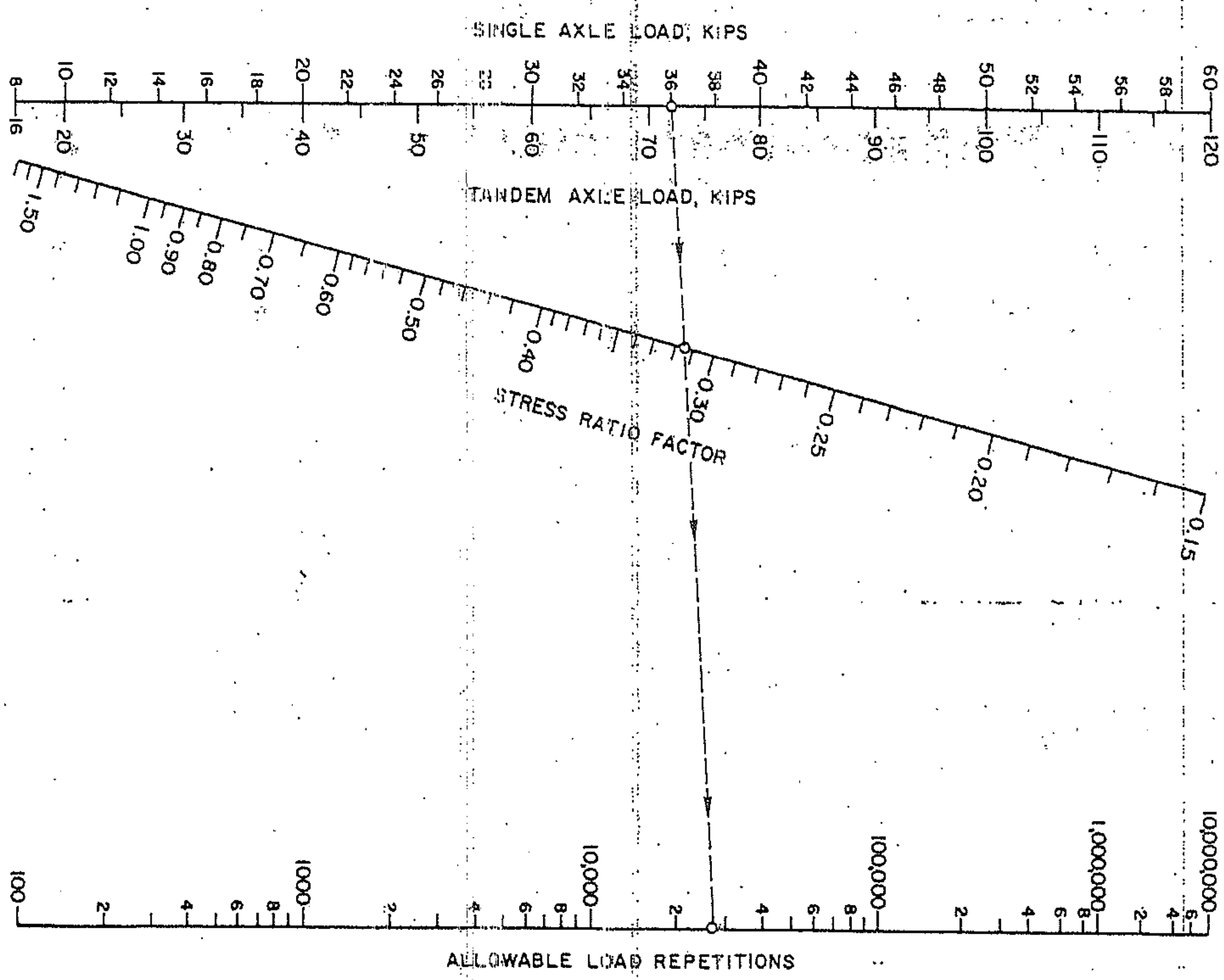


Figure 14-22 A nomograph for fatigue analysis—allowable load repetitions based on stress ratio factor (with and without concrete shoulder.) (Courtesy Portland Cement Association.)

Table 14-6 Erosion Factors for Aggregate-Interlock Joints (Single Axle/Tandem Axle)

Slab Thickness (in.)	k of Subgrade-Subbase, pci					
	$k = 50$ lb/in.^{3}	$k = 100$ lb/in.^{3}	$k = 200$ lb/in.^{3}	$k = 300$ lb/in.^{3}	$k = 500$ lb/in.^{3}	$k = 700$ lb/in.^{3}
4	3.94/4.03	3.91/3.95	3.88/3.89	3.86/3.86	3.82/3.83	3.77/3.80
4.5	3.79/3.91	3.76/3.82	3.73/3.75	3.71/3.72	3.68/3.68	3.64/3.65
5	3.66/3.81	3.63/3.72	3.60/3.64	3.58/3.60	3.55/3.55	3.52/3.52
5.5	3.54/3.72	3.51/3.62	3.48/3.53	3.46/3.49	3.43/3.44	3.41/3.40
6	3.44/3.64	3.40/3.53	3.37/3.44	3.35/3.40	3.32/3.34	3.30/3.30
6.5	3.34/3.56	3.30/3.46	3.26/3.36	3.25/3.31	3.22/3.25	3.20/3.21
7	3.26/3.49	3.21/3.39	3.17/3.29	3.15/3.24	3.13/3.17	3.11/3.13
7.5	3.18/3.43	3.13/3.32	3.09/3.22	3.07/3.17	3.04/3.10	3.02/3.06
8	3.11/3.37	3.05/3.26	3.01/3.16	2.99/3.10	2.96/3.03	2.94/2.99
8.5	3.04/3.32	2.98/3.21	2.93/3.10	2.91/3.04	2.88/2.97	2.87/2.93
9	2.98/3.27	2.91/3.16	2.86/3.05	2.84/2.99	2.81/2.92	2.79/2.87
9.5	2.92/3.22	2.85/3.11	2.80/3.00	2.77/2.94	2.75/2.86	2.73/2.81
10	2.86/3.18	2.79/3.06	2.74/2.95	2.71/2.89	2.68/2.81	2.66/2.76
10.5	2.81/3.14	2.74/3.02	2.68/2.91	2.65/2.84	2.62/2.76	2.60/2.72
11	2.77/3.10	2.69/2.98	2.63/2.86	2.60/2.80	2.57/2.72	2.54/2.67
11.5	2.72/3.06	2.64/2.94	2.58/2.82	2.55/2.76	2.51/2.68	2.49/2.63
12	2.68/3.03	2.60/2.90	2.53/2.78	2.50/2.72	2.46/2.64	2.44/2.59
12.5	2.64/2.99	2.55/2.87	2.48/2.75	2.45/2.68	2.41/2.60	2.39/2.55
13	2.60/2.96	2.51/2.83	2.44/2.71	2.40/2.65	2.36/2.56	2.34/2.51
13.5	2.56/2.93	2.47/2.80	2.40/2.68	2.36/2.61	2.32/2.53	2.30/2.48
14	2.53/2.90	2.44/2.77	2.36/2.65	2.32/2.58	2.28/2.50	2.25/2.44

Source: Thickness Design for Concrete Highway and Street Pavements, Portland Cement Association, Skokie, IL, 1984.

In the preceding example, we have illustrated the PCA design procedure for one set of conditions: plain joints, no subbase, and no concrete shoulder. Reference 17 provides similar illustrations and examples of other combinations of subbase support, shoulder treatment, and joint design.

PROBLEMS

1. A railroad culvert is being designed for a 64-hectare drainage area near St. Louis, Missouri. The time of concentration is 15 min. It is estimated that 70 percent of the rainfall will infiltrate the ground or evaporate or otherwise not show up as runoff. Estimate the runoff in cubic meters per second, assuming a 25-year recurrence interval.
2. A highway culvert is to be designed for a 95-acre drainage area near Chicago, Illinois. The time of concentration is estimated to be 18 min. The drainage area is flat residential with 60 percent impervious soil. Using a 10-year recurrence interval, estimate the runoff in cubic feet per second.
3. A 900-mm circular concrete culvert ($n = 0.013$) is to be placed under a highway. The estimated flow is 2.0 m^3/sec. The culvert is 30 m long and has a 45-degree beveled edge inlet. Determine the difference in elevation between the upstream and downstream water surfaces.

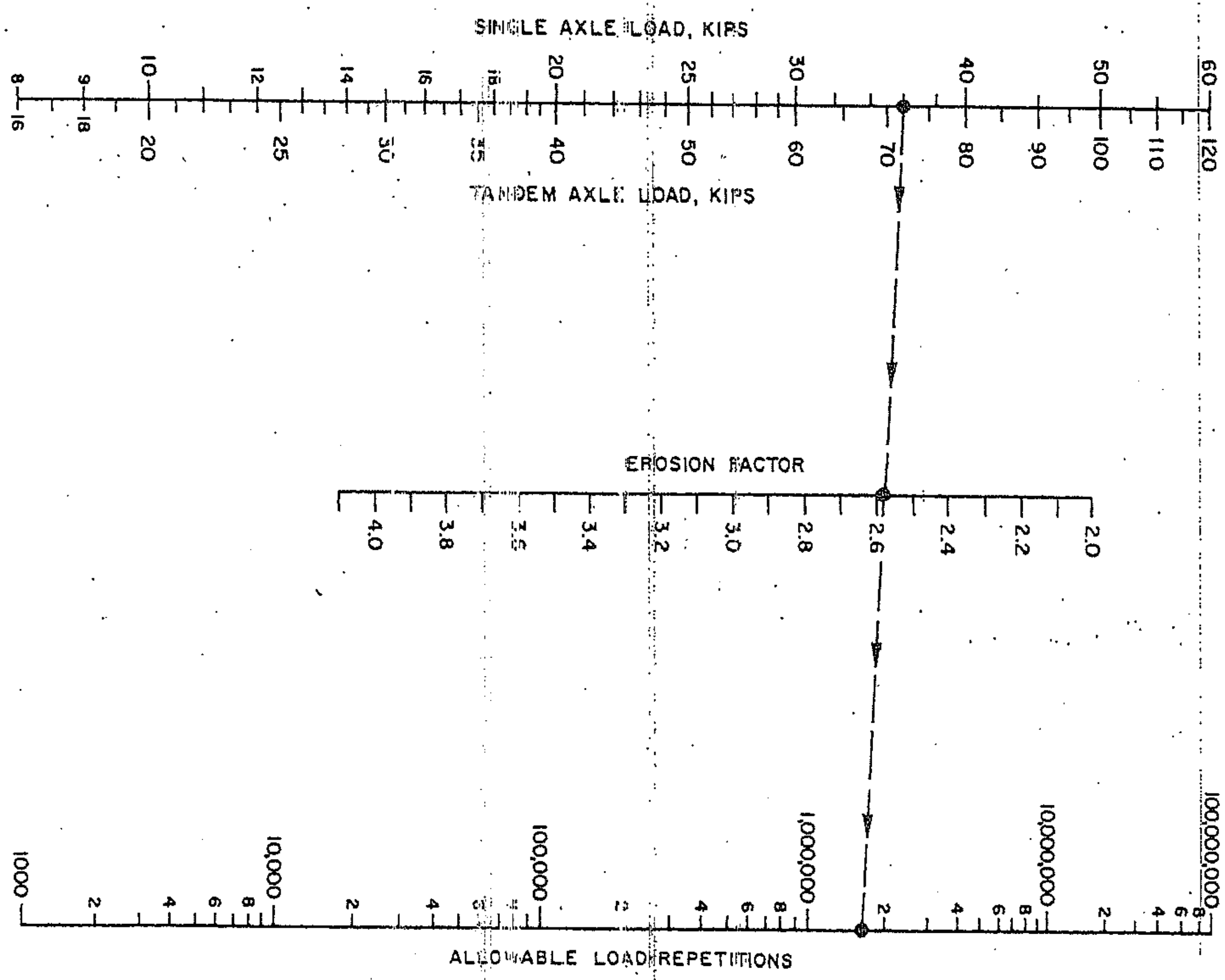


Figure 14-23 An erosion analysis nomograph—allowable load repetitions based on erosion factor without concrete shoulder. (Courtesy Portland Cement Association.)

4. A 2-ft flat-bottom channel lined with riprap with 2:1 side slopes has a slope of 0.4 percent, and the depth is 1.5 ft. Determine the quantity of flow in cubic meters per second.
5. A smooth concrete 2-ft flat-bottom channel with 2:1 side slopes has a slope of 0.3 percent, and the depth is 1.2 ft. Determine the quantity of flow in cubic feet per second.
6. Determine the depth and velocity of flow in a trapezoidal channel ($n = 0.03$) with 2:1 side slopes and a 2-ft bottom width, given a flow of 200 ft^3/sec and a slope of 2.0 percent. Determine the depth and velocity of flow if $n = 0.012$.
7. Given an allowable headwater depth of 2.5 m and a runoff of 1.0 m^3/sec, determine the required size of corrugated metal pipe and the actual headwater depth. Assume that the pipe has a mitered entrance that conforms to the slope.
8. Given an allowable headwater depth of 10 ft and a runoff of 150 ft^3/sec, determine the required size of circular corrugated metal pipe and the actual headwater depth. Assume that the pipe has a projecting entrance and will operate under inlet control.
9. Assuming a freehaul distance of 1000 m, cost of excavation of \$2.50/m^3, and price of overhaul of \$10/m^3-km, what is the limit of economical haul?
10. Given below are the ordinates of a portion of a mass diagram for a hypothetical highway project. No haulage is permitted back of station 20 or beyond station 40, the end of the project. The freehaul distance is 800 ft. Draw the mass diagram, and determine (a) the volume of excavation and embankment; (b) the quantities of overhaul (station yards), waste, and borrow; and (c) the direction of haul.

Station	Ordinate	Station	Ordinate	Station	Ordinate
20	0	27	−28,000	34	+4,000
21	−4,000	28	−32,000	35	+10,000
22	−8,000	29	−26,000	36	+8,000
23	−12,000	30	−20,000	37	+6,000
24	−16,000	31	−14,000	38	+4,000
25	−20,000	32	−8,000	39	+2,000
26	−24,000	33	−2,000	40	0

11. On the basis of the mass diagram from problem 10, make a sketch of the profile of the project showing the beginning points and relative heights of cuts and fills.
12. The design structural number of a certain pavement is SN = 4.00. A 4-in. asphaltic concrete surface has been chosen for the pavement. Crushed limestone is to be used for the base, and sand-gravel will serve as the subbase. The policy of the highway agency is to use a minimum base thickness of 6 in. What thickness should be used for the subbase?
13. Given the traffic roadway conditions described in Example 14-3, determine the thickness of concrete pavement required if the subgrade has a modulus of subgrade reaction $k = 50$ lb/in.^{3}

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Chapter 15

~~Standard~~ Station designs AMTRAK

Design of Land Transportation Terminals

Yards 494

Rail pass. stations 487

Terminals often constitute the most important component of a transport system. Terminal costs comprise a significant if not dominant portion of the total costs of transportation. An inadequately designed terminal facility may cause inordinate delays to the movement of passengers or freight and ultimately may contribute to the failure of the system to remain competitive with other companies or modes.

An average railway freight car travels only 92 km/day (57 miles/day), a journey requiring only about 1 hr [1]. During the other 23 hr of the day, the car sits idle at a yard or other terminal facility. Similarly, it is not unusual for ships, motor carriers, and aircraft to spend more than half of the time unproductively at a terminal facility.

The physical features of land transportation terminals vary a great deal depending on the transport mode, the type of commodity, and the amount of traffic it serves. Clearly, major design differences are created by the differing sizes and operational characteristics of the transporting vehicles. That trucks, buses, railways, and pipelines have inherently different terminal needs is so obvious that it hardly bears mentioning. Even essentially similar passenger modes such as commuter rail and intercity rail facilities vary significantly because of the different needs of the two types of travelers.

15-1. FUNCTIONS OF TERMINALS

Land transportation terminals may be required to perform eight basic functions:

1. *Traffic concentration* Passengers arriving in continuous flows are grouped into batch movements; small shipments of freight are grouped in larger units for more efficient handling.
2. *Processing* This function includes ticketing, checking in, and baggage handling for passengers and preparation of waybills and other procedures for freight.
3. *Classification and sorting* Passengers and freight units must be classified and sorted into groups according to destination and type of commodity.
4. *Loading and unloading* Passengers and freight must be moved from waiting rooms, loading platforms, temporary storage areas, and the like to the transportation vehicle at the origin, and the process must be reversed at the destination.
5. *Storage* Facilities for short-term storage such as waiting rooms for passengers and

transit sheds for freight commodities are required to permit loads to be assembled by concentration and classification.

6. *Traffic interchange* Passengers and freight arriving at a terminal often are destined for another location and must transfer to a similar or different mode of travel to complete the journey.
7. *Service availability* Terminals serve as an interface between the transport user and the carrier, making the transportation system and its services available to the shipper and the traveling public.
8. *Maintenance and servicing* Terminals often must include facilities for fueling, cleaning, inspection, and repair of vehicles.

15-2. THE NATURE OF THE TERMINAL PLANNING PROCESS

Although the major focus of this chapter is on terminal design, it seems appropriate to consider some of the approaches recommended for the planning of terminal facilities. The objective of the terminal planner is to define the elusive "optimum" design—one that is sufficient in size and complexity to provide a suitable level of service but not so elaborate that it could involve wasteful outlays for construction and operation of facilities that would be idle much of the time.

The planner must first forecast the future level of activity at the terminal: the number of passengers to be accommodated by passenger terminals as well as their patterns and modes of arrival and departure and their needs while at the terminal; the volume of freight, classified by commodity type, and again, the patterns and modes of shipment to and from the terminal. The procedures used for forecasting terminal demand vary a great deal depending upon terminal type and size. In certain instances, forecasts can be based on historical data, empirical studies, and extrapolation of trends. In forecasting passenger terminal requirements, planners may need to perform surveys of parkers and travelers to determine current travel deficiencies and desires. Models such as those described in Chapter 9 also may be helpful in estimating levels of passenger terminal activity. Planners of freight terminal facilities sometimes base forecasts on known or assumed relationships between the tonnage of freight and the volume of wholesale or retail sales, gross regional product, or some other measure of economic growth. Freight terminal planners may find it necessary to perform special studies of vehicle arrival rates and times, loading and unloading rates, processing procedures, and work habits and rules.

Usually, a terminal facility is designed to provide for 5 to 10 years in the future. However, forecasts in demand must appropriately account for fluctuations over time, including seasonal, daily, and hourly variations. In most instances, it is not advisable to design for the absolute peak-day or peak-hour demand expected over the forecast period. Adjustments should be made for a continuous peak period but not necessarily the highest day's or highest hour's activity. Rather, a typical peak-hour demand is usually chosen for passenger movements similar to the thirtieth highest hourly traffic volume recommended for highway design. A typical peak daily traffic is recommended for estimating the level of freight terminal activity.

If the number of vehicles or passengers arriving at a terminal were precisely predictable, if their times of arrival could be scheduled, and if their service times could be anticipated, the problem of planning the most economical facilities could possibly be determined by an elementary arithmetical analysis. However, arrivals at a terminal are not regular but tend to be characterized by chance variability about the mean arrival rate. Furthermore, the time required for processing or servicing vehicles or passengers at a terminal is not constant but also tends to have a random component.

Because of the probabilistic nature of terminal operations, an elementary arithmetical analysis of such problems is not possible. For the same reason, certain aspects of terminal operations are subject to analysis by queuing or waiting-line theory.

15-3. QUEUING THEORY

An extensive treatment of queuing theory is beyond the scope of this book. This chapter will sketch some of the most elementary applications of these methods to terminal planning. Generally speaking, queuing theory is most useful for analyses of the behavior of simple waiting lines or for studies of some component of more complex operations.

Several characteristics of a queue must be known or assumed if queuing problems are to be solved analytically:

1. The mean rate at which units (vehicles or people) arrive for service and the probability distribution of the arrivals
2. The mean service rate and the probability distribution of the services
3. The number of channels or servers (e.g., truck loading spots, toll booths, etc.) and whether the channels are arranged parallel (as a toll booth) or in a series (as in a vehicle repair facility)
4. The queue discipline, the order in which arriving units will be served

Both arrivals and service times are random variables. Arrivals are discrete random variables, and service times are continuous random variables. It is often appropriate to describe units arriving at a terminal by the Poisson probability distribution:

$$P(n) = \frac{(\lambda t)^n e^{-\lambda t}}{n!} \tag{15-1}$$

where

$P(n)$ = probability of n arrivals in a period t
λ = mean arrival rate or volume
e = Napierian logarithmic base

It may be advantageous to focus on the time intervals or headways between successive arrivals rather than on the number of arrivals occurring during a stated interval of time. For a Poisson process, it can be shown that the probability density function[1] of interarrival times is

$$f(t) = \lambda e^{-\lambda t} \tag{15-2}$$

This equation, known as the negative exponential distribution, commonly is expressed as a cumulative distribution function, expressing the probability of a headway h being greater than or equal to t:

$$P(h \geq t) = \int_t^\infty f(t) = e^{-\lambda t} \tag{15-3}$$

[1] A probability density function is a nonnegative function $f(t)$ whose integral over the entire t-axis is unity

Table 15-2 Relationships for Multichannel Queues with Poisson Arrivals and Negative Exponential Service Times

1. Probability of having exactly zero units[a] in the system:

$$P(0) = \left\{\left[\sum_{n=0}^{k-1} \frac{1}{n!}\left(\frac{\lambda}{\mu}\right)^n\right] + \frac{1}{k!}\left(\frac{\lambda}{\mu}\right)^k \frac{k\mu}{k\mu - \lambda}\right\}^{-1}$$

2. Probability of having exactly n units in the system:
   (a) For $n < k$

$$P(n) = \frac{1}{n!}\left(\frac{\lambda}{\mu}\right)^n P(0)$$

   (b) For $n \geq k$

$$P(n) = \frac{1}{k!k^{n-k}}\left(\frac{\lambda}{\mu}\right)^n P(0)$$

3. Average number of units in the system:

$$\overline{n} = \frac{\lambda\mu(\lambda/\mu)^k}{(k-1)!(k\mu - \lambda)^2} P(0) + \frac{\lambda}{\mu}$$

4. Average length of queue:

$$\overline{m} = \frac{\lambda\mu(\lambda/\mu)^k}{(k-1)!(k\mu - \lambda)^2} P(0)$$

5. Average waiting time of an arrival:

$$\overline{w} = \frac{\mu(\lambda/\mu)^k}{(k-1)!(k\mu - \lambda)^2} P(0)$$

5. Average time an arrival spends in the system:

$$\overline{v} = \frac{\mu(\lambda/\mu)^k}{(k-1)!(k\mu - \lambda)^2} P_0 + \frac{1}{\mu}$$

where
λ = average number of arrivals per unit of time
μ = average service rate for each channel, number of units per unit of time
k = average number of channels or service stations

[a]Vehicles or people.

should be made regarding how arrivals and services are to be modeled, the nature of the queue discipline, and what measures of effectiveness are to be used. It may be appropriate to describe arrival and service distributions by analytical queuing models such as Eqs. 15-1, 15-2, and 15-3, but field studies may be needed to justify the use of these equations or else to indicate another approach. In certain instances, empirical data can be utilized directly by means of a Monte Carlo model. See Section 15-5 for a discussion of the Monte Carlo technique. To facilitate the writing of the computer program, it is recommended that a block or functional diagram be prepared showing the sequencing and interrelationships of events.

3. *Validation of the model* It is always advisable to observe the behavior of the model and compare typical results from the model (e.g., average queue lengths, waiting times, etc.) to empirical data. Only then can one be confident that the model reasonably describes the system being studied.
4. *Design of experiments* Once the model is validated, a series of experiments should be designed to test the behavior of the terminal system under various levels of traffic demand and combinations of design layouts and operating conditions.
5. *Evaluation of results* Simulation models provide a means of studying the consequences of design and management decisions on the operational efficiency of a terminal. Typically, the analyst wishes to evaluate the effect of such decisions on queue lengths, delays, and other measures of effectiveness and the impact of long queues, delays, and the like on the system, its users, and its environs.

15-5. EXAMPLE OF SIMULATION OF WAREHOUSE LOADING FACILITIES

Consider the example of a simulation study to determine the requirements for warehouse dock facilities for trucks [2]. A chain of department stores decided to consolidate its warehouse facilities at a single location. The problem was to determine the desired number of loading docks required to handle the volume of truck traffic formerly accommodated by the three separate locations plus that occasioned by future growth.

Empirical studies of truck arrivals and servicing revealed that truck arrival rates were not significantly different among half-hour intervals during the morning and that variations within half-hour periods followed a pattern closely corresponding to the Poisson law.

Since the truck arrival rates did not directly apply to the consolidated warehouse, it was necessary to break them down into elements that could be identified as being present or absent in the new operational scheme and to recombine the elements appropriately. Allowance for future growth of truck traffic was based on the assumption that it would be proportional to the expected future dollar volume of the retail stores involved.

Two types of trucks were identified: vendor trucks that would unload only and delivery trucks that would load only. The vendor trucks were divided into two classes each having service times described by an exponential distribution but with different average servicing times. Specifically, 75 percent of the vendor trucks required an average servicing time for 10 min, and the remaining 25 percent of the trucks required 45 min.

It was found that the service times of the delivery trucks could be modeled by a shifted exponential distribution that accounted for an average of 18 min each driver spent with paperwork and preloading cargo assembly.

A Monte Carlo simulation procedure was employed. The procedure for each truck involved the assignment of a time of arrival and a servicing time. These quantities were selected randomly from stocks of numbers (frequently distributions) that reflected the properties established by the empirical studies. For each specified number of service docks, the movement of trucks through the docking system was simulated, and the length of waiting line and the delay until service were determined. Repeated simulations of a day's operations were conducted, and the results were averaged for each set of conditions. Figure 15-1 illustrates one of the results from this example simulation study.

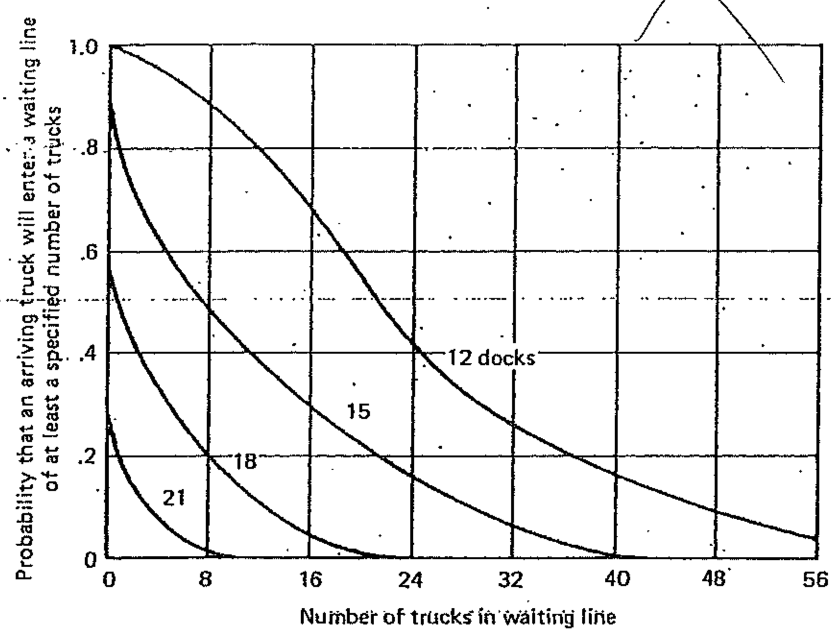


Figure 15-1 Results from example simulation study.

TERMINAL DESIGN CONTROLS AND CRITERIA

In the remainder of this chapter, design controls and criteria for the principal land transportation terminals are described in the order outlined here:

- Passenger terminals
  - Automobile parking facilities
  - Bus terminals
  - Rail passenger stations
  - Urban rail transit stations
- Freight terminals
  - Truck terminals
  - Rail classification yards
  - Pipeline tank farms

Automobile Parking Facilities

Widespread parking problems exist in central business districts, on college campuses, at shopping centers, and in industrial parks and other highly developed areas. Shortages of parking spaces in close proximity to major traffic generators have worsened as automobile registrations have increased and more and more people have chosen to travel by automobile. Parking problems have been aggravated in many cities as curb parking has been eliminated to improve flow.

The procedures for conducting parking surveys are described in Chapter 8. In the following paragraphs, the principles for the location, layout, and design of curb and off-street parking are discussed [3].

15-6. CURB PARKING

Curb parking tends to seriously impede traffic flow and contribute to conflicts and crashes. Many highway and traffic engineers therefore recommend that curb parking be prohibited along major streets. It should also be prohibited within bus stops and pedestrian crosswalks, adjacent to fire plugs, and in the vicinity of intersections, alleys, and driveways. Where permitted, curb parking should be regulated to minimize its effects on accidents and congestion and to ensure that available parking spaces are used appropriately and efficiently.

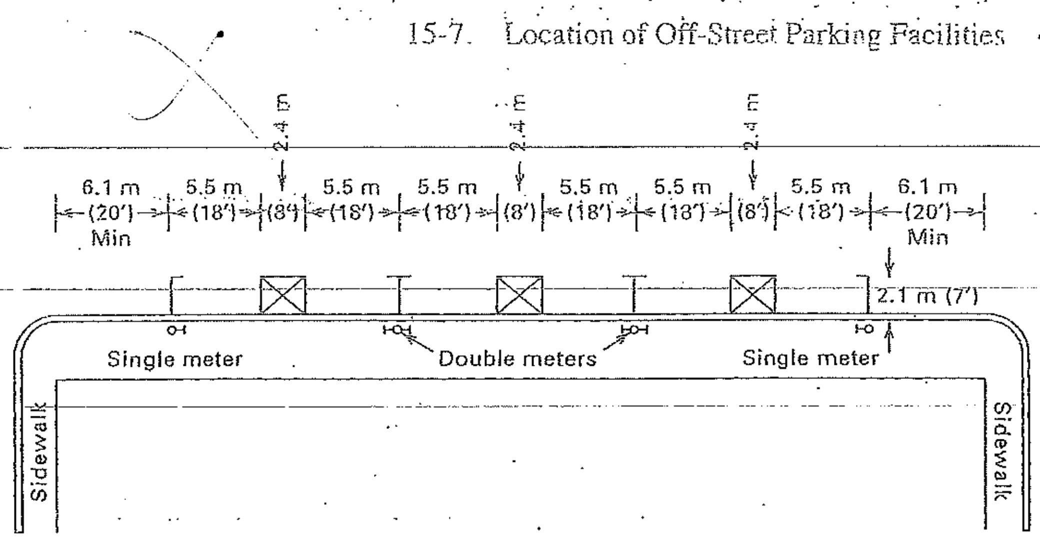


Figure 15-2 Paired curb parking layout. (Courtesy Transportation Research Board.)

From a safety and operational viewpoint, parallel parking is preferred over angle parking, and curb parking functions best when stalls are marked properly.[2] Properly marked parking stalls tend to make it easier for drivers to park, lessening the likelihood of congestion and crashes. One study [4] found a 43 percent reduction in the average time required to park after suitably marked parking stalls were painted. Stalls are especially recommended for locations where there is great demand for curb parking spaces and where parking meters are used.

Three basic types of parallel stalls are used [4]: (1) end stalls, (2) interior stalls, and (3) paired parking stalls. End stalls are situated adjacent to intersections, alleys, driveways, and other restricted areas. End stalls are typically 6.1 m (20 ft) in length. Interior stalls are usually 6.7 m (22 ft) long, providing about 1.4 m (4.5 ft) between adjacent cars for maneuvering. As Fig. 15-2 illustrates, paired parking consists of pairs of contiguous 5.5-m (18-ft) stalls separated by an 2.4-m (8-ft) open space that may be used for maneuvering. In this arrangement, the open spaces must be marked clearly to prevent drivers from parking there. Paired parking is used frequently in conjunction with double parking meters, that is, two meters installed on a single post.

Curb parking stalls are designated by white lines extending out from the curb a distance, typically of 2.1 m (7 ft). The end row of curb parking spaces normally is marked with an inverted L-shaped line, and interior stalls are designated by a T-shaped line.

Curb parking stalls should not be placed closer than 6.1 m (20 ft) to the nearest sidewalk edge at nonsignalized intersections. At signalized intersections, a clearance to the sidewalk edge of 15.2 m (50 ft) and preferably 30.5 m (100 ft) should be observed. Parking stalls should be placed not closer than 4.6 m (15 ft) away from fire hydrants and driveways [4].

15-7. LOCATION OF OFF-STREET PARKING FACILITIES

One of the most important aspects of parking facility planning and design is the choice of a site. Experience has shown that an improperly located lot or garage is likely to fail or have limited use, even if located only a few blocks away from the center of parking demand.

Most parkers are reluctant to walk even short distances from the parking location to their ultimate destination. The maximum walking distance a parker will tolerate depends

[2]An exception to the rule on angle parking may be allowed where through traffic is excluded and a street is dedicated fully to parking and access.

on trip purpose, city size, and the cost of parking. Office workers may be willing to walk a maximum distance of about 600 m (2000 ft) if the rates are attractively low [4]. Short-term parkers such as shoppers will usually resist walking more than one or two blocks. Generally speaking, the smaller the city and the higher the parking cost, the less distance a parker is willing to walk.

Preferably, parking lots and garages should be located on or near major arterials; garages should have access to two or more streets. It is also desirable that both lots and garages be accessed by right-hand-turning movements.

Safe and convenient pedestrian access to parking facilities should be provided. To alleviate parkers' fears for their personal security, pedestrian walkways should be well lighted, suitably marked, and free of blind corners. Most pedestrians prefer overhead bridges to tunnels. To the extent feasible, pedestrian conflicts with vehicular traffic should be avoided.

In summary, parkers prefer parking facilities near their destination, that are easily accessible, that can be used without fear for one's personal safety, and that will cost little or no money.

15-8. LAYOUT OF PARKING LOTS AND GARAGES

Ideally, a parking lot or garage should be rectangular with cars parked on both sides of access aisles. Parking sites should be wide enough to provide two or more bays approximately 18 m (60 ft) wide. Relatively long aisles, 76 m (250 ft) or longer, function well. Aisles running with the long dimension of the lot or garage provide a better search pattern and yield more spaces per unit area.

Ninety-degree parking is generally used for two-way traffic. Right-angled parking tends to require slightly less area per parked car than do other configurations.

Parking stalls oriented at angles of 45 to 75 degrees to the aisles are often used with one-way circulation. Where such an arrangement is used to serve a building along the end of the parking lot, drivers must circulate at least once next to the building during entry or exit, causing more vehicle-pedestrian conflict than the two-way, 90-degree layout.

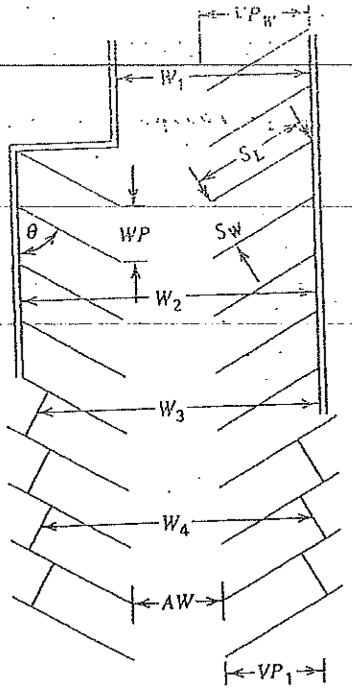
Where space permits, it is desirable to provide pedestrian sidewalks between adjacent rows of parked cars. When sidewalks are provided, it is usually necessary to install wheel stops to prevent vehicle encroachment. For 90-degree pull-in parking, the wheel-stop setback from the edge of the curb should be about 0.75 m (2.5 ft). For back-in parking, a setback of at least 1.2 m (4.0 ft) is recommended [4].

Stall widths of 2.6 to 2.7 m (8.5 to 9.0 ft) commonly are used for parking facilities in the United States. Widths of 2.9 m (9.5 ft) or even 3.0 m (10.0 ft) are sometimes used for supermarket lots and other areas where packages are being placed in cars [4]. Stall depths of 5.4 to 5.6 m (18.0 to 18.5 ft) are used most often. For future layouts designers will need to consider the effects of vehicle downscaling and use smaller stalls.

The most common and preferred layout pattern is the bumper-to-bumper interlocked pattern illustrated by Fig. 15-3. The herringbone, or nested, interlock is sometimes used with 45-degree parking. In that configuration, the bumper of one car faces the fender of another car necessitating the installation of wheel stops and increasing the probability of vehicular damage. Parking layout dimensions for stalls arranged at various angles are given by the table that accompanies Fig. 15-3. The best parking layout for a given site, usually determined by a process of trial, will depend primarily on:

1. The size and shape of the available area
2. The type of facility (self-park, attendant)
3. The type of parker (short-term, long-term)
4. The type of operation (pull-in, back-in, one-way, two-way, etc.)

- θ Parking angle
- W_1 Parking module width (wall to wall), single-loaded aisle
- W_2 Parking module width (wall to wall), double-loaded aisle
- W_3 Parking module width (wall to interlock), double loaded
- W_4 Parking module width (interlock to interlock), double-loaded aisle
- AW Aisle width
- WP Stall width parallel to aisle
- VP_1 Projected vehicle length from interlock
- VP_W Projected vehicle length from wall measured perpendicular to aisle
- S_L Stall length
- S_W Stall width



Parking Angle and Projected Vehicle Length		Stall Widths		Aisle Widths	Module Widths			
		S_w	WP	AW	W_1	W_2	W_3	W_4
Large-car Vehicle 1956 mm by 5461 mm (77" by 215")								
90°	VP_w – 5.61 (18.42) VP_1 – 5.61 (18.42)	2.59 (8.50)	2.59 (8.50)	7.33 (24.04)	12.94 (42.46)	18.56 (60.88)	18.56 (60.88)	18.56 (60.88)
75°	VP_w – 5.93 (19.45) VP_1 – 5.68 (18.62)	2.59 (8.50)	2.68 (8.80)	6.45 (21.17)	12.38 (40.62)	18.31 (60.07)	18.06 (59.24)	17.80 (58.41)
60°	VP_w – 5.84 (19.16) VP_1 – 5.35 (17.55)	2.59 (8.50)	2.99 (9.82)	4.29 (14.09)	10.13 (33.25)	15.97 (52.41)	15.48 (50.80)	14.99 (49.19)
45°	VP_w – 5.25 (17.21) VP_1 – 4.55 (14.94)	2.59 (8.50)	3.66 (12.02)	3.35 (11.0)	8.60 (28.21)	13.84 (45.42)	13.15 (43.15)	12.46 (40.88)
Small-car Vehicle 1676 mm by 4445 mm (66" by 175")[a]								
90°	VP_w – 4.60 (15.08) VP_1 – 4.60 (15.08)	2.29 (7.50)	2.29 (7.50)	6.79 (22.27)	11.38 (37.35)	15.98 (52.43)	15.98 (52.43)	15.98 (52.43)

[a]Small care spaces normally are considered only for 90-degree layouts.

Figure 15-3 Parking stall layout elements expressed in meters (feet). (Adapted from *Parking*, Eno Foundation for Transportation, 1990.)

15.9. PARKING GARAGE DESIGN CRITERIA

Special design criteria for parking garages, abstracted from reference 5, are given in the following paragraphs.

Single entrances and exits, with multiple lanes, are preferable to several openings. Entrances and exits should be located away from street intersections to prevent traffic congestion. Lane widths of 3.6 to 4.3 m (12 to 14 ft) typically are used for entrances and exits. Lanes should be tapered to a width of 2.7 to 3.0 m (9 to 10 ft) for approaches to ticket dispensers and cashiers' booths.

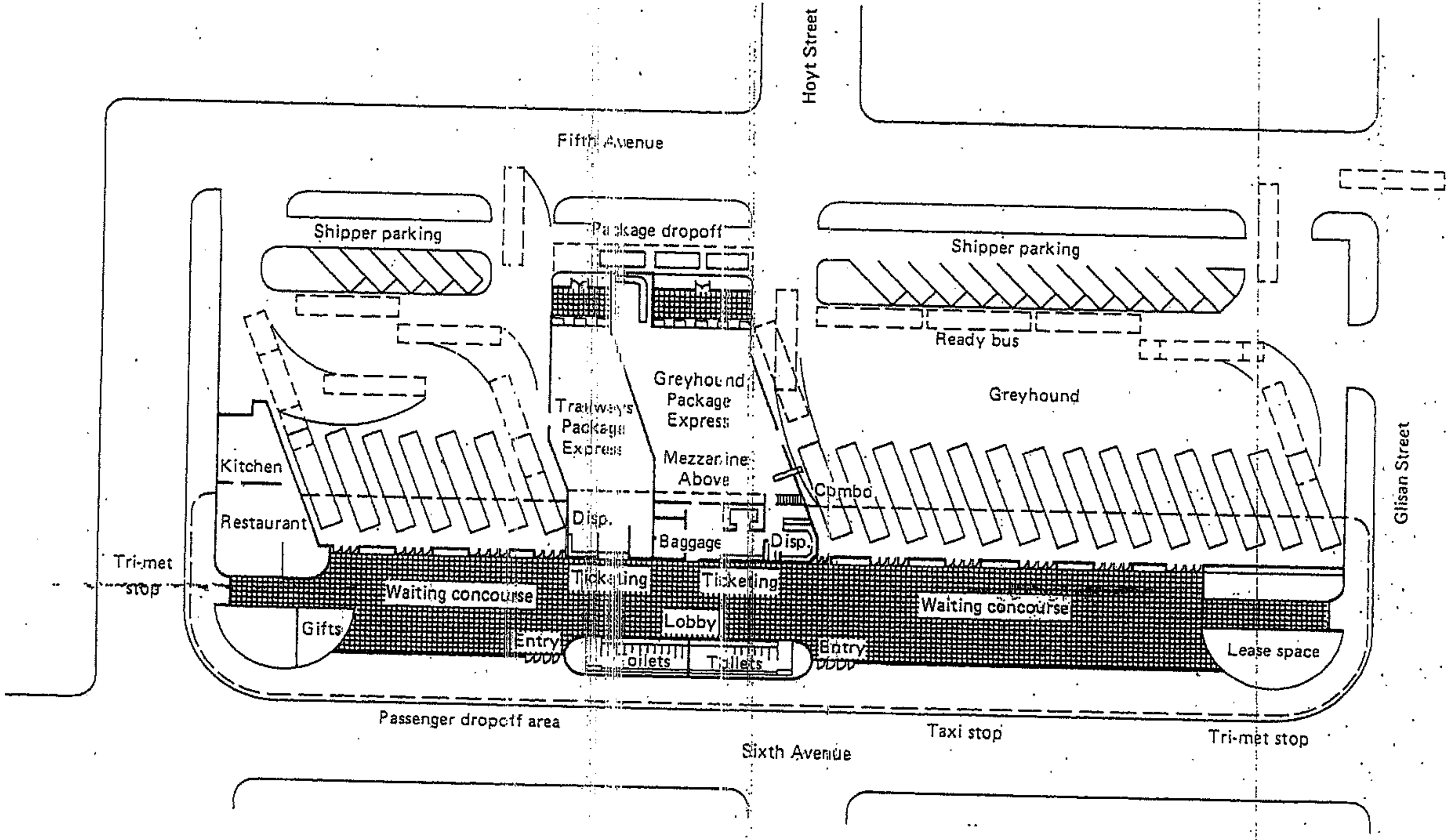


Figure 15-4 Typical layout of bus transportation center. (Courtesy Trailways Bus System.)

When self-parking is used, there is little need for reservoir space at the entrance. Studies indicate that the capacity of an entrance lane with an automatic ticket dispenser is approximately 400 vehicles/hr. One entrance lane for every 300 to 500 spaces usually is provided.

If attendant parking is used, a storage reservoir will be needed. The reservoir space required will depend on the passenger unloading time and the time required for parking each vehicle as well as the arrival rate.

If possible, clear span construction should be provided with column spacing equal to the unit parking depth (module width). A clear ceiling height of 2.1 m (7 ft), preferably 2.3 m (7.5 ft), should be provided.

Vehicular access between floors can be provided by sloped floors or by ramps. For self-park facilities, the floor slopes should not exceed 3 to 4 percent. Floor slopes up to 10 percent can be used for attendant park facilities; ramp slopes should preferably not exceed 10 percent. Straight ramps should be at least 2.7 m (9 ft) wide; curved ramps are usually at least 3.6 to 4.0 m (12 to 13 ft) in width. A minimum radius of ramp curvature of 9.1 m (30 ft) is recommended, measured at the face of the outer curb of the inside lane.

Counterclockwise circulation is preferred for parking garages. When helical ramps are used, the down ramp should be placed inside and the up ramp should be outside [3].

15-10. BUS TERMINALS

The layout and design of intercity bus terminals vary a great deal in size and complexity depending on the number of passengers to be accommodated and whether the facilities are to serve one, two, or more carriers. Figure 15-4 shows a typical layout for a transportation center designed to serve two carriers. The layout features common waiting area, restaurant, and toilets but has separate ticketing, passenger loading, and baggage-handling areas.

Angle parking is commonly provided for the buses, typically arranged as shown by Fig. 15-5. The spatial requirements depend primarily on the size, physical features, and operating characteristics of the buses and the angle of parking. Fig. 15-5 gives recommended loading platform dimensions required for a 40-ft by 8.5-ft bus using a 45-degree parking angle. To facilitate efficient baggage handling and passenger loading, a minimum clearance of 6 ft between buses is desirable. A protective canopy overhang should be provided and should extend at least beyond the loading door of parked buses.

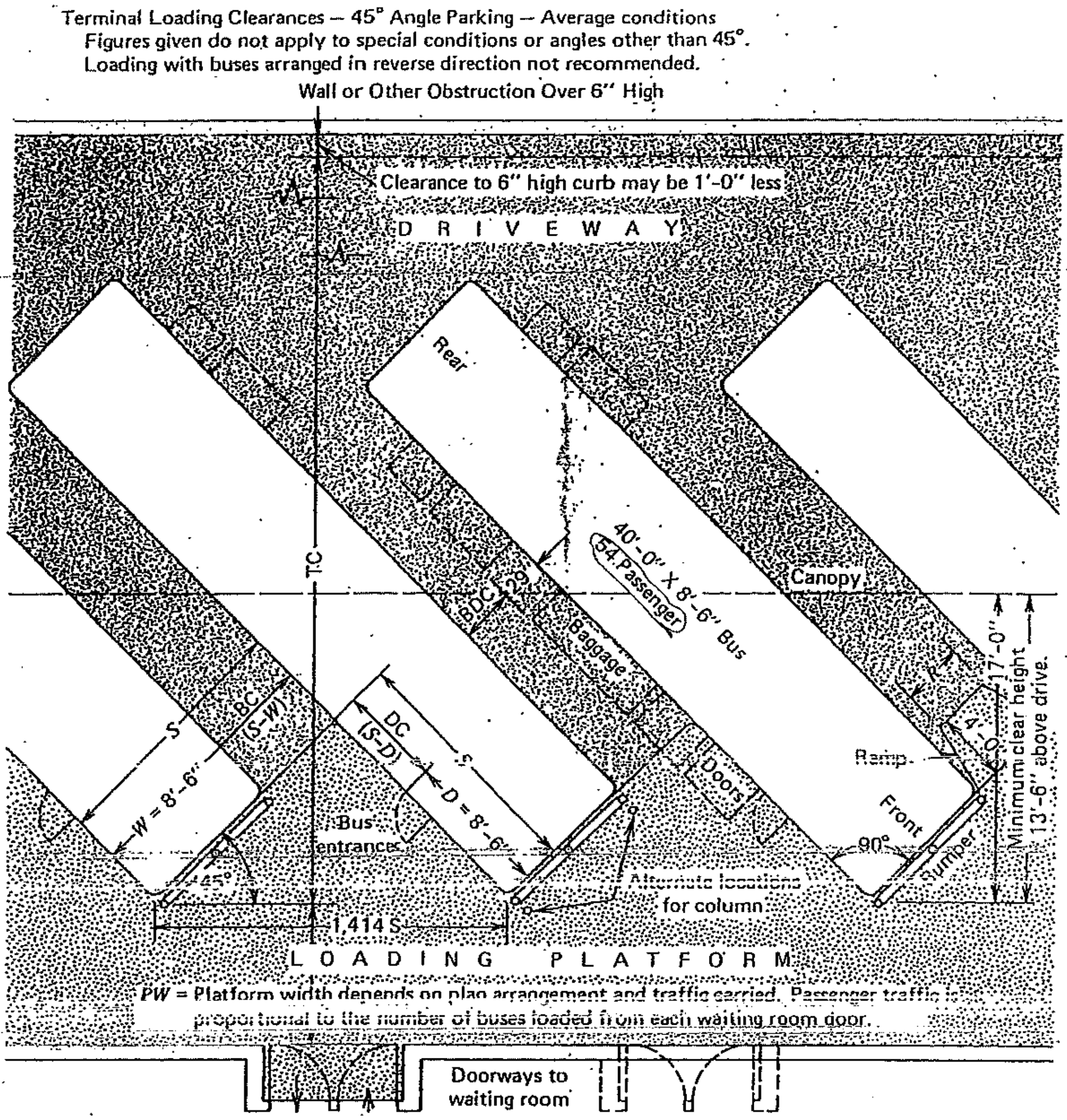
Suitable provisions must be made for package express handling, temporary storage, pickup, and dropoff.

All new or substantially renovated terminals must conform to the minimum design standards in the national standard specifications for making buildings and facilities accessible to and usable by the physically handicapped. These specifications require the provision of reserved parking spaces for the handicapped, entrances usable by persons in wheelchairs, ramps sloped not steeper than 8.33 percent with handrails at least on one side, and a number of other requirements [6].

The amount of space required for waiting area, ticket counters, toilets, and other public areas is based on empirical relationships between spatial needs and typical peak-hour passenger volumes. For further guidance on this aspect of the problem, the reader should refer to Chapter 17.

15-11. RAIL PASSENGER STATIONS

Rail passenger travel in the United States declined sharply following World War II, reaching a low of 254 million passengers transported in 1973. That figure, which included urban commuters, represented 31 percent of all passengers traveling by public carrier between U.S. cities; however, excluding urban commuters, rail travel accounted for only 0.7 percent of the



Tabulation:

Clearance		For reserve parking (S = Spacing center to center of buses)		For passenger loading			
		11'	12'	14'	15'	16'	17'
TC = Recommended minimum turning clearance for typical positions*	to curb	63'-6"	60'-6"	55'-3"	52'-9"	52'-0"	51'-6"
	to wall	64'-6"	61'-6"	56'-3"	53'-9"	53'-0"	52'-6"
BC = Clearance between buses; *DC* = Door clearance		2'-6"	3'-6"	5'-6"	6'-6"	7'-6"	8'-6"
BDC = Baggage door clearance		1"	1'-1"	3'-1"	4'-1"	5'-1"	6'-1"
R = Ramp width		1'-	2'-6"	3'-10"	4'-6"	5'-0"	5'-6"

*For clearance at intermediate spacings see graph. Special conditions require special clearances. When buses enter with right turn from street, first few loading positions will require additional turning clearance.

Figure 15-5 Parking layout for buses at terminal. (Courtesy Greyhound Lines.)

Table 15-3 Features of AMTRAK's Standard Station Designs

Design	Design Volumes, Typical Peak-Hour Passengers	Number of Ticket Counters	Overall Station Area m²	ft²	Auto Parking, Number of Cars
300A	300–1000	4	1,672	18,000	225
150B	150–350	3	764	8,220	110
50C	50–175	2	186	2,000	30
24D	25–75	1	107	1,150	20
E	<25	0	22	240	15

Source: Standard Stations Program, Executive Summary, AMTRAK, Washington, DC, 1978.

total intercity travel expressed in passenger miles. Since 1973, there have been increases in both the number of passengers traveling by rail and the passenger mileage, reaching approximately 338 million passengers and 13.8 billion passenger miles in 1995. Yet AMTRAK, the National Railroad Passenger Corporation established to provide improved rail passenger service, continues to experience annual operating deficits of about $1 billion [7].

Although few new railroad passenger stations are being constructed, AMTRAK has developed a standard stations program that includes five classes of facilities. The salient features of these designs are given in Table 15-3.

15-12. RAIL STATION DESIGN

A rail station includes the station tracks, the platforms, connecting thoroughfares (passageways, ramps, stairways, etc.), and the terminal building.

With respect to the track layout, two types of passenger stations may be used:

1. ***Through station*** With this common arrangement, the trains stop on a mainline track while passengers board and disembark. This type of station is satisfactory where traffic is light and only through service is provided. Where rail traffic is heavy, additional platform tracks are provided alongside main tracks, allowing freight trains and nonstop passenger trains to bypass the platforms. In large urban centers, multiple platform tracks are provided, allowing trains serving a variety of origin and destination points to be in the station at the same time. Trains enter these tracks through "throat tracks," with 2.5 to 3.0 station tracks being provided for each throat track.
2. ***Stub station*** With the stub station, trains reverse direction of travel when leaving the station. This type normally would be used for the end of a line. Stub stations in large urban areas have multiple platforms accessed through throat tracks. In such stations, crossovers and tracks between the platforms are provided so that locomotives can depart for servicing while the train is discharging passengers and baggage. Loop tracks in a few large terminals (e.g., Grand Central Terminal in New York) allow trains to be turned without reversing and crossing the terminal throat tracks.

Passenger platforms and corridors at downtown stations should be at least 4.5 m (15 ft) in width. A minimum width of 1.8 m (6 ft) is recommended for suburban or rural stations [8].

Spatial requirements for passengers in the main building are based on empirical studies, and the areas required for various functions in the station usually are based on the number of typical peak-hour passengers. These requirements are not unlike those described in Chapter 17 for air passengers, but the facilities are generally much smaller in scale. Typical requirements for various facilities in rail stations are given in Table 15-4.

Table 15-4 Space Requirements for Various Facilities in Rail Stations

Passenger Areas		
Waiting areas for peak-hour passengers	<150	1.3 m^2 (14 ft^2)/passenger[a]
	150–500	1.1 m^2 (12 ft^2)/passenger[a]
	>500	0.9 m^2 (10 ft^2)/passenger[a]
Baggage area		0.65–0.85 m^2 (7–9 ft^2)/passenger
Coffee shop/eating areas for peak-hour passengers	<300	Vending machines
	>300	0.28–0.37 m^2 (3–4 ft^2) per passenger in dining/lunch room. One seat for every 7–10 passengers.
Kitchen for peak-hour passengers	>300	46–74 m^2 (500–800 ft^2)
Ticket counter		
Ticket positions, passengers/hour		25–35
Length per agent position		2.3 m (7.5 ft)
Length of queuing line		4.6 m (15 ft)
Depth, with conveyor		2.4 m (8 ft)
Depth, without conveyor		1.5 m (5 ft)
Restrooms		
Toilet fixtures		2 + PHP[b]/50
Lavatories		2 + PHP/75
Corridor widths, one-way		0.015 × PHP, m (0.05 × PHP, ft)
Corridor widths, two-way		0.024 × PHP, m (0.08 × PHP, ft)
Boarding		
Gate queue		9 m (30 ft)
Area		0.56 × PHP, m^2 (6 × PHP, ft^2)
Employee Areas		
Employee lounge		9.3 m^2 + 0.9 m^2 per employee on duty (100 ft^2 + 10 ft^2 per employee on duty)
Cash accounting office		5.6 m^2 + 3.7 m^2 per employee above 2 (60 ft^2 + 40 ft^2 per employee above 2)
Station services office		11.1 m^2 (120 ft^2)
Supervisor's office		7.4 m^2 (80 ft^2)
Station manager's office		11.1 m^2 (120 ft^2)
Secretarial area		7.4 m^2 (80 ft^2)
Red cap ready room		9.3 m^2 + 0.9 m^2 for each employee >5 (100 ft^2 + 10 ft^2 for each employee >5)

[a] Plus visitors.

[b] Peak-hour passengers.

Source: Standard Stations Program, AMTRAK, Washington, DC 1978; *Manual for Railway Engineering,* American Railway Engineering Association, Washington, DC 1995.

The number of ticket counter positions will depend on the mode of operation. At positions dedicated to ticket sales only, approximately 35 passengers per hour can be accommodated. Agents accepting baggage only can process about 55 passengers per hour. In most AMTRAK stations it can be assumed that about 25 percent of the peak-hour passengers have been ticketed [9].

Rail passenger stations and the facilities in the stations must be accessible to and usable by the physically handicapped.

Urban Rail Transit Stations

Urban rail transit stations differ from conventional rail passenger stations in several respects. For example, in urban rail transit stations:

1. Little or no provision must be made for the handling of baggage.
2. There are higher densities of passengers and heavier flows requiring more rapid entry and egress. The vehicles therefore are designed with wide, automatic doors, and platforms are built at the level of the vehicle floors.
3. The trains operate at short headways, typically 10 min during off-peak periods and as little as 90 sec during peak periods. There is relatively little need for separate waiting areas with seats or benches for waiting patrons.
4. Automated ticketing and fare collection systems are used and must be provided for in the station design.
5. Since many urban rail stations are built as subways or elevated facilities, special attention must be paid to vertical circulation and the planning and design of street-level entrances and exits.
6. Whenever urban rail transit stations are located in high-crime areas, special security measures such as monitored closed-circuit television must be employed to ensure public safety and to allay patrons' fears of crime.
7. At suburban rail transit stations, facilities may need to be provided for feeder bus loading and unloading, passenger car loading and unloading (kiss-and-ride), and short-term and long-term parking.

Figure 15-6 illustrates a modern urban rail transit station.

15-13. TYPICAL RAIL TRANSIT STATION DESIGN PROCEDURES AND CRITERIA

In the following paragraphs, design procedures and criteria for the Metropolitan Atlanta Rapid Transit Authority (MARTA) system are briefly described [10]. The approach used for that system is presented as typical of that employed for urban rail transit stations.

Estimates of station patronage were made for the A.M. peak hour. The P.M. peak-hour patronage was assumed to be equal to but in opposite direction to that of the A.M. peak hour.

Station elements were sized on the basis of the *ultimate station capacity*, which was 150 percent of the station patronage. However, parking lot capacity was based on the requirements projected in the patronage analysis.

Certain elements of design were based on the peak 5-min patronage, taken to be 12.5 percent of the ultimate station capacity. Others were based on the peak-minute patronage, assumed to be 2.5 percent of the ultimate station capacity.

MARTA stations have been designed on the basis of levels of service as recommended by John Fruin and as described in Section 6-8. During normal operating conditions, the levels of service were assumed to be between C and D. For emergency conditions,

Figure 15-6 A modern urban rail transit station. (Courtesy American Consulting Engineers Council.)

MARTA designers assumed that the patron flow is all exiting the station, and the level of service would be between D and E.

The sizing of station platforms was based on the *station accumulation,* which was assumed to be equal to the maximum number of patrons waiting on the platform if a train is up to 90 sec late during the ultimate peak 5-min period.

Two basic types of platforms were used: a center platform between tracks and a side platform at one side of a single track or on each side of an adjacent pair of tracks.

MARTA platforms were designed for lengths of at least 183 m (600 ft), the length of the eight-car trains expected to operate ultimately at peak hours. At above-ground stations, wind and weather screens were provided along approximately 90 m (300 ft) of each side platform to shelter waiting patrons from the elements.

The platforms were sized to allow 0.74 m^2 (8 ft^2) of unobstructed walking surface per patron of the station accumulation on the platform. Additional widths of 0.3 m (1 ft) from walls and 0.4 m (1.5 ft) from the platform edge were provided.

The designers adhered to several fundamental principles of station circulation, namely:

1. Right-hand flows were preferred.
2. Whenever possible, cross flows were avoided.
3. Passenger flows moving in opposite directions were separated.
4. Dead-end conditions were avoided.
5. Design features were chosen and arranged so as to eliminate or minimize delays.

The part of the rail transit station that facilitates the movement of large numbers of people in and out of the stations is known as the concourse. It functions as a transition area between the points of entry into the station and the access ways to the train platforms. It provides space for a variety of functions, including fare collection, directional and informational signing, and a location for a comfortable waiting area, especially during off-peak hours. The concourse may be located at street level, platform level, or a mezzanine area between these levels.

The design of the concourse and access ways is highly dependent on the requirements for fire and panic safety. The MARTA design standards allow for exiting requirements to accommodate the maximum plausible emergency conditions, specifically, a station load equal to the ultimate passenger load of a train during the peak 15 min plus the ultimate peak-minute entraining patron load awaiting that train on the platform. The standards specify that at least two platform exits must be provided a minimum of 30.5 m (100 ft) apart. The standards further provide [10]:

1. There shall be a minimum of two exits from each concourse located as far apart as possible.
2. In subway stations, no point on the platform(s) or concourse shall be located more than 60 m (200 ft) from an exit.
3. In above-ground stations, no point on the platform(s) or concourse shall be located more than 90 m (300 ft) from an exit.

Sufficient exit lanes were required to evacuate the exiting occupant load in 4 min or less. Emergency exiting capacities were evaluated on the basis of 610-mm (24-in.) lane widths, a lane capacity of 40 persons per minute, and a travel speed of 61 m/min (200 ft/min). In calculating the number of exit lanes available, 0.3 m (1 ft) was deducted from the width of exit corridors and ramps to allow for side walls, and 0.6 m (2 ft) was deducted to allow for a buffer zone at the platform edge.

MARTA stations were designed to ensure accessibility and usability for the handicapped. To this end, MARTA stations are designed with:

1. At least one primary entrance accessible to individuals in wheelchairs
2. Walks at least 1219 mm (48 in.) wide with gradients not more than 5 percent
3. Ramp slopes not more than 8.33 percent with hand rails on at least one side 813 mm (32 in.) in height
4. Wall fixtures appropriately mounted to make them reachable by the handicapped
5. Designated automobile parking spaces for use by individuals with physical disabilities

15-14. TRUCK TERMINALS

The design of a truck terminal depends on the area to be served, the volume of freight to be handled, and the level of service to be provided. An adequate number of doors must be provided to assure that the terminal meets its schedules and freight-handling requirements. The doors must have suitable dimensions and allow for proper clearance and maneuvering space for equipment. Adequate yard space must also be provided for parking and access to the loading platform and operating areas [11].

A minimum width of 3.35 m (11 ft), preferably 3.65 m (12 ft), is recommended for each spot in order to allow proper clearances on each side of 2.6-m-(8.5-ft-) wide vehicles. The dock approach length is the length of the largest tractor-trailer combination using the dock plus the apron length necessary to maneuver the vehicle into and out of the parking spot. Generally speaking, the dock approach should be at least twice the length of the longest tractor-trailer combination. Table 15-5 shows the minimum recommended apron and dock approach lengths assuming 3.65-m (12-ft) widths for dock spots. If 3.35-m (11-ft) widths are used, the apron lengths shown should be increased by about 0.6 m (2 ft).

The apron dock approach area should be nearly level. Although most trucks are designed to negotiate a 15 percent grade, the startup grade for pulling away from the dock should not exceed 3 percent [11].

The dock height should be approximately equal to the bed height of the vehicles using the dock. Typically, trailer bed heights vary from about 1219 to 1320 mm (48 to 52 in.), while pickup and delivery vehicle bed heights range from about 1118 to 1219 mm (44 to 48 in.). In order to ensure that vehicle doors can be opened and closed, it is best to select a dock height that is too low rather than too high.

Table 15-5 Minimum Recommended Apron Lengths and Dock Approach Lengths for Truck Terminals

Overall Length of Tractor-Trailer		Apron Length		Dock Approach Length	
m	ft	m	ft	m	ft
12.2	40	13.1	43	25.3	83
13.7	45	14.9	49	28.7	94
15.2	50	17.4	57	32.6	107
16.8	55	18.9	62	35.7	117
18.2	60	21.0	69	39.3	129
19.8	65	22.9	75	42.7	140

Source: Adapted from *Shipper-Motor Carrier Dock Planning Manual,* The Operations Council, The American Trucking Associations, Washington, DC, 1970.

Vertical clearances of at least 4.6 m (15 ft) are recommended in order to accommodate 4.1-m-(13.5-ft-) high trailers.

Traffic circulation plans should encourage vehicles to circulate in a counterclockwise direction. For two-way traffic, service roads should be a minimum of 7.0 m (23 ft), preferably 7.3 m (24 ft), in width. For one-way roads, the minimum widths on straight sections should be 3.65 m (12 ft). On curves, the pavement must be widened to conform to the wheel paths of the turning vehicles.

The width of parking spaces for trucks should be about 3.65 m (12 ft), and the length should be the vehicle length plus 20 percent.

15-15. RAILROAD FREIGHT TERMINALS AND YARDS

Railroad yards provide shops and facilities for the maintenance of the rolling stock and storage of idle cars. The most important need, however, is for classification—the making up of trains for the distribution of freight to various parts of the country.

A railroad freight terminal facility usually contains at least three components:

1. A receiving yard, where incoming trains are directed from the main line and are stored temporarily before being sorted and classified
2. The classification yard, where cars are sorted and classified into blocks of common destination
3. The departure yard, where the sorted blocks of cars are made into trains and stored while arrangements are made for mainline movement

A large terminal facility may also contain a repair yard and a local yard, the latter being for classification of cars scheduled for local deliveries. A typical layout of a large railroad freight terminal facility is shown in Fig. 15-7. Small yards may be considerably less elaborate, containing only one general yard with certain tracks being assigned for receiving and departure activities.

The layout of the various yards depends primarily on the dimensions of the cars and locomotives, the length of the trains, the volume of traffic, and the rate at which cars are processed. Generally speaking, a minimum of 15.2 m (50 ft) per car should be allowed for all freight car tracks except for repair tracks [which require 16.8 m (55 ft) per uncoupled car] and for special-equipment tracks [8]. Body tracks[3] should be spaced not less

[3]Body tracks are those tracks in the yard where cars are parked, while ladder tracks provide access to the yard.

than 4.3 m (14 ft) center to center. When parallel to a main track, the first body track should be spaced not less than 6.1 m (20 ft) from that track center to center. Parallel ladder tracks are usually spaced not closer than 5.5 m (18 ft) center to center.

A sufficient number of receiving tracks should be provided so that there will be one available whenever an arriving train needs to enter the yard. The type of classification yard and the number of tracks depend on the volume and scheduling of traffic to be handled. Flat yards are sometimes used where the number of switching cuts per train is small. In such facilities, the sorting of cars is performed by switching engines at a rate of 30 to 60 cars/hr [1].

At large and busy terminals, the sorting of cars is accomplished by gravity or hump switching. The cars are pushed by an engine to the top of a hump located between the receiving and classification yards. The gradients leading to the classification yard are designed carefully so that each car will roll to and stop at the far end of the yard or will roll to a coupling at a speed of not more than 6.4 km/hr (4 mph). The speed of a car on a grade can be calculated at any point by use of fundamental energy relationships. The change in velocity that occurs depends on the magnitude and length of slope. The calculations, described in more detail in reference 8, are similar to those for calculating the velocity of a freely falling body. Suitable allowance must be made for energy losses due to rolling resistance and that due to the rotation of the wheels.

In modern classification yards, retarders are placed along the tracks leading to the classification yards to control the speed of the entering cars. These devices bear against the flanges of the wheels to slow or stop the car as it moves through the retarder. The amount of braking pressure that will be applied by the retarder may be controlled by an operator in a nearby tower by means of a push-button system. In newer yards the devices may be controlled automatically by computerized systems that measure the weight and speed of each car and calculate the needed retardation after appropriately accounting for variations due to wind resistance, rolling resistance, and coupling or stopping distance.

Classification rates may vary from 100 to 300 cars/hr depending on the degree of automation used. Figure 15-8 shows a classification yard at the Union Pacific Railroad Company's Bailey Yard.

Departure yards should have a sufficient number of tracks so that there will be one available for assembling a departing train when needed. The length of the departure tracks will depend on the length of the completed trains including the assisting locomotives. To avoid excessive starting resistance, gradients adverse to the forward movement of the trains should be avoided.

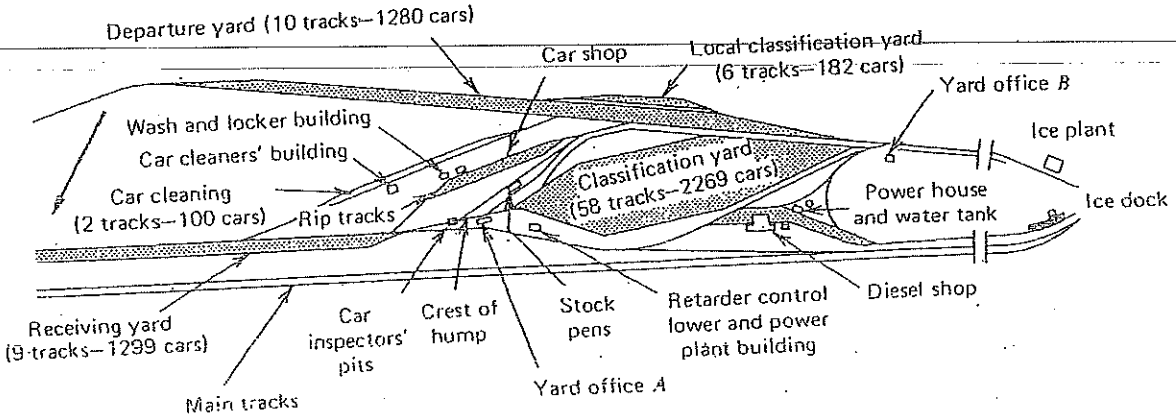


Figure 15-7 Plan of a classifications yard. (Source: *An Introduction to Transportation and Engineering*, Second Edition, by Wm. W. Hay, John Wiley & Sons, 1977.)

Figure 15-8 Union Pacific Railroad Company, Bailey Yard, North Platte, Nebraska.

15-16. TANK FARMS

In pipeline transportation, the terminals take the form of groupings of strategically arranged storage tanks called tank farms. Depending on the nature of the product being transported, a wide variety of storage tanks are used. For example, certain gases such as propane must be stored in high-pressure tanks. Here, the discussion is limited to terminal facilities for liquid petroleum products that can be stored in atmospheric tanks, that is, at atmospheric pressure.

Three general types of tanks are used: fixed roof tanks, floating roof tanks, and tanks that combine features of both fixed and floating roof tanks. Most liquids can be stored in floating roof tanks. Liquids that have a high vapor pressure are usually stored in floating roof tanks to prevent excessive evaporation. Some pipeline companies prefer hybrid tanks that have a fixed roof combined with an internal lightweight floating pan to control evaporation.

Generally, storage tanks are built above ground, although special circumstances may dictate their placement underground. For example, they are often built underground at major airports to facilitate safe aircraft movements.

The minimum size of tank used depends on the minimum batch size transported by the pipeline. The available storage capacity of a tank farm is determined by the number and size of all the tanks, while the required storage capacity is a function of the turnover.

Pipeline companies that transport flammable and combustible liquids conform to strict standards for the location of storage tanks with respect to public areas. The companies generally follow the recommendations of reference 12, which gives minimum distances from storage tanks to adjacent property lines and public transport ways.

Adjoining property and waterways are protected further by means of impounding by diking around the tanks. Reference 12 recommends that the ground surface around a tank

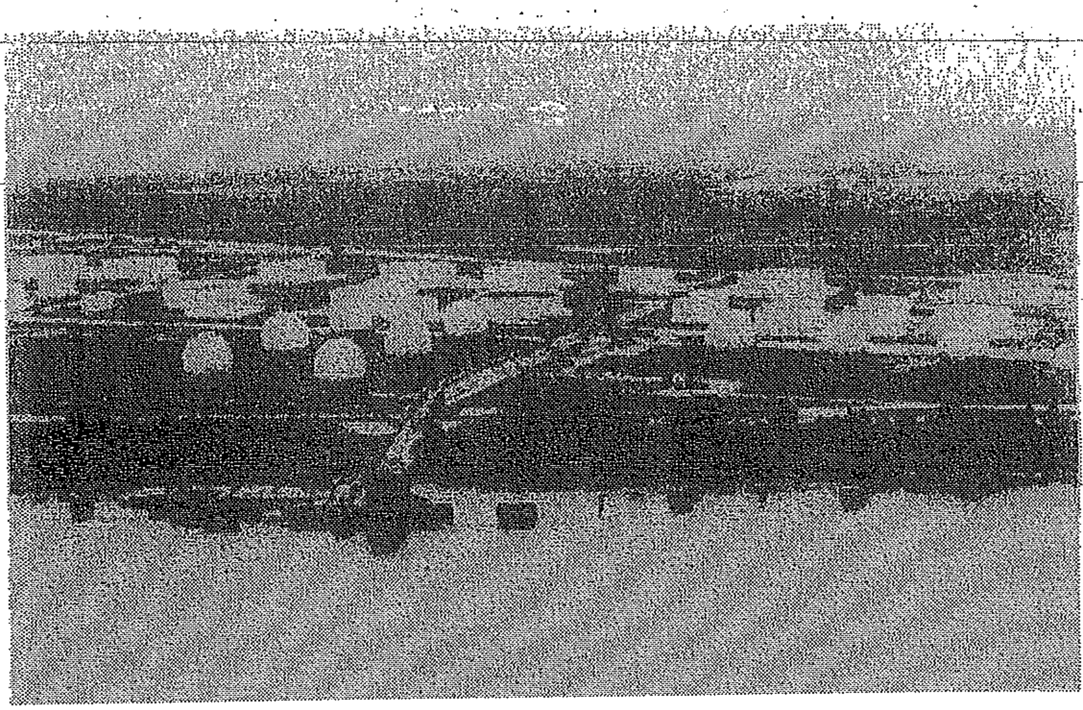


Figure 15-9 Aerial view of a tank farm. (Courtesy Chicago Bridge and Iron Co., Inc.)

be sloped at least 1 percent away from the tank for at least 15 m (50 ft) or to the base of the dike, whichever is less. The volumetric capacity of the diked area should not be less than the greatest amount of liquid that can be released from the largest tank within the diked area, assuming a full tank [12].

The dike walls may be made of earth, steel, concrete, or solid masonry but must be liquid tight and able to withstand a full hydrostatic head. Earthen walls 1 m (3 ft) or more in height should have a flat section at the top at least 0.6 m (2 ft) wide.

Storage tanks usually are built directly on the ground or on foundations made of concrete, masonry, piling, or steel. The foundations should be designed so as to minimize the likelihood of uneven settlement of the tank. The tank may need to be specially treated to resist corrosion.

A typical tank farm is shown in Fig. 15-9.

PROBLEMS

1. How many cars can be placed in a commercial parking lot on land that is 40 m wide (street frontage) and 100 m deep? Make a sketch showing the design using SI (metric) units. Assume two driveways, one-way circulation, and self-parking.
2. How many cars could be placed in a commercial parking lot 140 ft wide (street frontage) and 240 ft deep? Make a sketch showing the recommended arrangement. Assume two driveways, one-way circulation, and self-parking.
3. Vehicles arrive at a toll booth at an entrance to a state park at a rate of 240 per hour. One attendant is on duty and serves the vehicles at a rate of one every 12 sec.
   a. Estimate the percent of time that the attendant will be idle.
   b. What is the average number of vehicles in the queue (including the one being served)?
   c. Estimate the average waiting time of an arrival.

4. The park described in problem 3 has a reservoir space for only three vehicles plus the vehicle being served. If more than three vehicles join the queue, the traffic backs up and blocks traffic on a busy state highway.
   a. What is the probability that traffic will be blocked?
   b. What is the probability that traffic will be blocked if two attendants are used? (*Note:* In this state, five vehicles can be in the system without traffic being blocked.)
   c. What percent of the time will one or both of the attendants be idle if two attendants are used?
5. The following are the predicted volumes of entering and departing automobiles at a proposed parking garage during a typical weekday. Draw a cumulative flow diagram of vehicles in the garage and indicate the minimum number of spaces required.

Period	Arriving Volume, (vehicles/hr)	Departing Volume, (vehicles, hr)
6–7 A.M.	14	6
7–8	16	7
8–9	120	10
9–10	138	15
10–10	43	23
11–12	20	29
Noon–1 P.M.	15	25
1–2	14	13
2–3	14	15
3–4	10	120
4–5	6	100
5–6	4	26

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Part V

Design of Air Transportation Facilities

Here we focus more closely on the air transportation mode and describe airport master planning and airport layout and design. The three chapters in this part of the book discuss the location of airports and the determination of runway orientation and length, the function and layout of airport aprons and terminal buildings, the geometric design of runways and taxiways, the design of airport drainage systems, and airport marking and lighting systems.

Airport Demand
509

RWY capacities
528

Select Airport Site
504

RWY config.
520

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Chapter 16

Airport Planning and Layout

16-1. INTRODUCTION

It was midmorning on a windy beach at Kitty Hawk, North Carolina, on December 17, 1903, when a fragile-looking two-winged craft with a man at the control was propelled on a little trolley along a wooden rail. It rose from the rail, surged forward into the wind, and settled back into the sand. Man's first flight in a heavier-than-air craft had lasted 12 sec and covered a distance of 37 m (120 ft), less than the wing span of a modern wide-bodied aircraft. On that morning, Wilbur and Orville Wright were to make three additional flights, the longest lasting for almost a minute. The air age had begun, and the world would never be the same.

Man's ancient aspirations to fly had been, in Wilbur Wright's words, "handed down to us by our ancestors who, in their gruelling travels across trackless lands in prehistoric times, looked enviously on the birds soaring freely through space, at full speed, above all obstacles, on the infinite highway of the air" [1].

The Wright's success had been preceded by centuries in which inaccurate and often ludicrous theories of flight were proposed, and abortive and sometimes disastrous attempts were made to leap and glide and soar.

A groundswell of interest in air transportation developed in 1783 when Joseph and Etienne Montgolfier demonstrated in Annonay, France, that man could travel by balloons filled with hot air. Later that same year, a French physicist, J. A. C. Charles, made a successful flight in a balloon filled with hydrogen.

Still later, when it became possible to steer these *montogolfières* and *charlières,* they came to be known as dirigibles. Strong interest in transportation by the slow and cumbersome dirigibles continued until the 1930s, when several spectacular disasters occurred to the lighter-than-air craft. These dramatic tragedies, along with progress in heavier-than-air flight technology, resulted in the assignment of the dirigible to very limited and specialized uses.

The Wright brothers' flight came at a time when there was arduous and seemingly frenetic activity by other aerial pioneers. The conviction that humanity was on the brink of successful heavier-than-air flight encouraged widespread study and experiment.

The Englishman George Cayley has been called the father of aerial navigation. His experiments during the first half of the nineteenth century, with small- and full-scale gliders, demonstrated the feasibility of flight in heavier-than-air craft. In 1866, another Englishman, F. H. Wenham, in a report to the first meeting of the Aeronautical Society of Great Britain, did much to advance the state of knowledge of aerodynamics and the design of wings.

A German, Otto Lilienthal, made over 2000 successful glides during the last decade of the nineteenth century, some of which covered several hundred feet. In 1894, Octave Chanute, a successful civil engineer in the United States, who was spurred by Lilienthal's successful experiments, published a historical account of man's attempts to fly. He later designed gliders on his own, using his knowledge of bridge building to improve their structural design. Chanute developed a close friendship with Orville and Wilbur Wright and was a source of help and encouragement during their years of experiments at Kitty Hawk.

The airplane made a spectacular, if not decisive, contribution to the outcome of World War I, and after the war more serious attention was given to airplanes as an effective means of transporting people and goods.

Acceptance of air transportation was increased by Charles Lindbergh's dramatic solo flight from New York to Paris in 1927. Air transportation was nearing the threshold of maturity at the advent of World War II, and few can dispute its vital contribution to the war effort. Figures 16-1 and 16-2 encapsulate the very rapid advance in aviation with one lifetime.

16-2. GROWTH OF AVIATION ACTIVITY

The growth of aviation activity since World War II has been unprecedented and extraordinary. United States air travel grew from 4.3 billion passenger miles in 1945 to 403.2 bil-

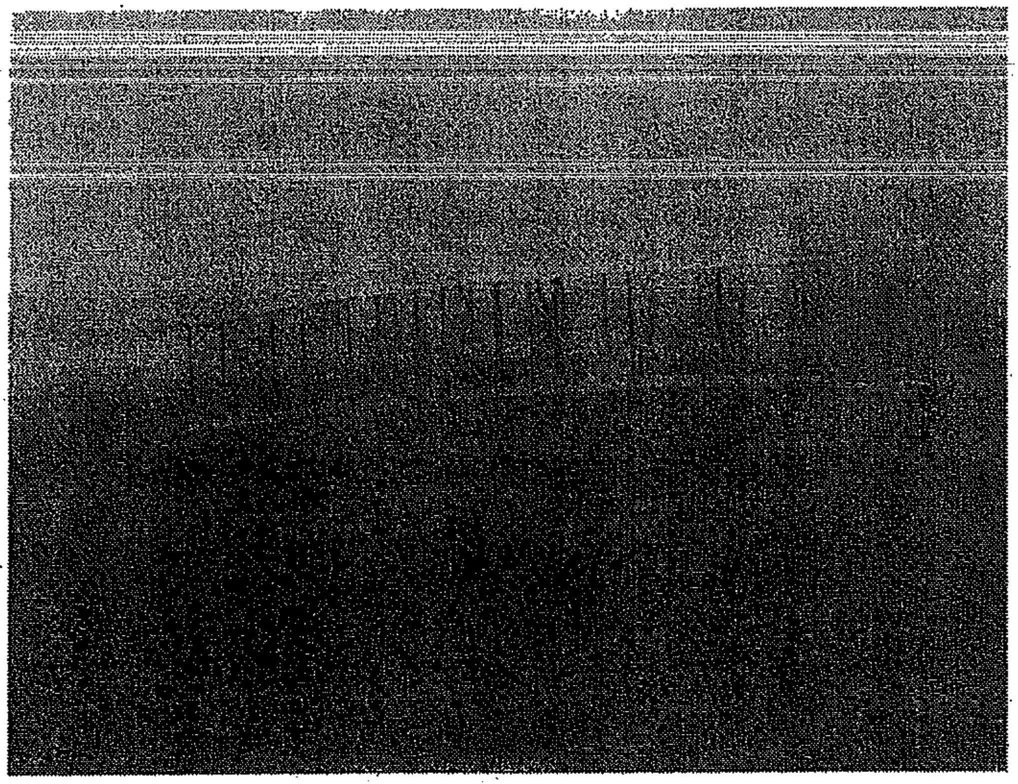


Figure 16-1 The first heavier-than-air flight, December 17, 1903. (Courtesy The Library of Congress.)

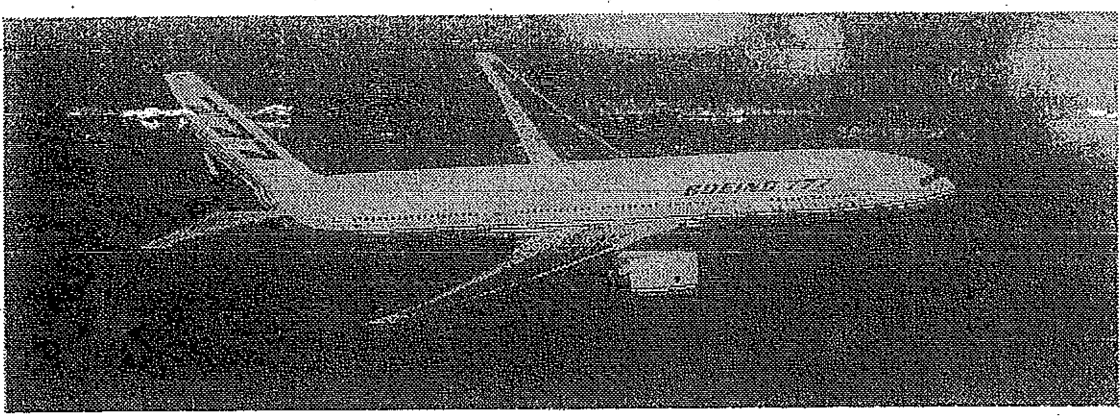


Figure 16-2 The Boeing 777 aircraft in flight. (Courtesy Boeing Corporation.)

lion in 1995. From 1975 to 1995 intercity travel (in terms of passenger miles) by air carrier nearly tripled, and travel by public air carriers now represents 17.1 percent of all intercity travel [2]. During that period, intercity travel by private aircraft remained practically constant, amounting to 11.3 billion passenger miles. Air freight has grown even faster than passenger travel, increasing an average of more than 6 percent annually since 1975 [2].

Trends in the factors that are accepted to be indicators of air travel activity (population, wealth, education, etc.) indicate that air travel will continue to grow in the years ahead. However, as the air transportation industry matures, it is likely that the rapid growth rate will stabilize to a lower steadier rate that is close to the rate of population growth, modified to some degree by socioeconomic changes [3].

16-3. NATURE OF THE PROBLEM

In the pages that follow, some of the problems and techniques of planning and designing an airport will be described. The remainder of this chapter will be devoted to a discussion of airport planning, site selection, runway orientation, obstruction clearance standards, and typical runway configurations. Chapter 17 will be concerned with the planning and design of the terminal area, while Chapter 18 will relate more specific design criteria and procedures for the runway and taxiway system.

Although these topics will be discussed separately, it should be recognized that there is an interaction between the various components of the problem and that goals and requirements of one component of the problem will often conflict with the needs of another. The overall task is an iterative one that seeks an optimum airport plan and design that best satisfies the various needs, constraints, and controls of the system.

16-4. AIRPORT DEMAND

Before engineering plans and designs for a new airport or improvements to an existing facility are made, a study should be made to determine the extent of future needs for airport facilities. Airport demand studies include forecasts of annual, peak-day, and peak-hour volumes of passengers and aircraft, types of forecast usage (i.e., business, commercial, passenger and freight, pleasure, etc.), as well as factors concerning size of community to be served and economic and population growth trends.

While the detailed techniques for making these forecasts are beyond the scope of this text, the following general observations and relationships are given as a matter of interest.

Experience has shown that a community's aviation activity is primarily dependent on (1) the population and population density of the city or region being served, (2) the economic character of the city, (3) the proximity of the airport to other airports, and (4) whether the airport acts as a hub for air transport routes.

Excluding hubbing activities, annual passenger enplanements and aircraft operations are directly related to city size; however, two cities approximately equal in population may have significantly different airport needs due to differences in the social and economic character of the city. Generally speaking, industrial cities tend to generate much less aviation activity than centers of government, education, and finance.

Airport demand studies must take into account the existence of nearby airports that may compete for passengers and freight. Passengers may elect to drive great distances to a competing airport in order to take advantage of cheaper fares, nonstop connections, or a more attractive schedule of flights. For example, a survey of Columbus, Georgia, travel agencies revealed that about 45 percent of airline passengers from that city elected to drive 100 miles to Atlanta's Hartsfield Airport in lieu of originating their air travel at the Columbus Metropolitan Airport.

A variety of approaches are used to forecast aviation demand. Such forecasts may be based on:

1. The judgment of the forecaster.
2. Predictions of knowledgeable persons from airline companies or the aircraft manufacturing industry.
3. National forecasts such as reference 4.
4. Analytical models that relate air travel to some combinations of variables such as air fares, travel time, city population, and social and economic characteristics of a city or pair of cities [5]. Other models called *time series models* based on the passage of time may also be used. Such models are conceptually similar to the planning models for surface transportation described in Chapter 9.

Reference can also be made to the *National Plan of Integrated Airport Systems* (NPIAS), a publication prepared and maintained by the Federal Aviation Administration that lists recommendations for future airport needs to promote the development of an adequate national system of airports. Airport development projects must first be included in NPIAS before being considered for federal-aid airport funds.

Once forecasts of annual air passengers are made, these must be converted to a peak hourly flow by using empirically based relationships, for example [3]:

10

$$\text{Average monthly passengers} = 0.08417 \times \text{annual passenger flow}$$

$$\text{Average daily passengers} = 0.03226 \times \text{average monthly flow}$$

$$\text{Peak-day flow} = 1.26 \times \text{average daily flow}$$

$$\text{Peak-hour flow} = 0.0917 \times \text{peak daily flow.}$$

Peak-hour passenger flows can be used to estimate peak hourly aircraft movements using estimates of average passenger load per aircraft.

16-5. SELECTION OF AN AIRPORT SITE

Perhaps the single most important aspect of the planning and design of airports is the selection of an airport site. Mistakes made in this phase of the airport development program can result in the failure or early obsolescence of the facility.

Several contemporary trends have complicated the problem of selection of an airport site:

1. Urban sprawl has occurred around most U.S. cities and has been accompanied by increasing scarcity of land and rising land costs.
2. Faster and larger aircraft appear at the facility, requiring longer runways and more aircraft service space along with increased automobile parking and circulation space.
3. The requirements and desires of the public regarding air passenger services have become more elaborate and sophisticated.
4. Increased aviation activity generates increased noise and other detrimental environmental impacts.

Guidelines regarding procedures for making a study of alternate airport sites along with a discussion of the major factors influencing site selection have been promulgated by the FAA [6]. While the critical investigation and location of airport sites is a responsibility of the airport sponsor,[1] FAA endorsement of the site is required if federal aid is contemplated. An airspace review by the FAA is required in any event.

After airport needs have been established, the FAA recommends that the following procedure be followed in selecting an airport site.

Desk Study of Area. Before a field investigation is made, a great deal can be learned from a desk study that includes:

1. A review of existing comprehensive land use plans and other community and area plans.
2. An analysis of available wind data to determine the desired runway orientation. (This is discussed in Section 16-6.)
3. A study of National Geodetic Survey quadrangle sheets, road maps, and aeronautical charts to select feasible sites for further evaluation.
4. A study of general land costs in the areas of interest.

In this study, special attention should be given to the location of other airports and land transportation facilities, obstructions, topographic features, and atmospheric peculiarities.

Physical Inspections. Actually, two physical field inspections should be made of the potential sites—preliminary and final:

1. The preliminary inspection should be made jointly by sponsor representatives of all existing airports and potential sites. After these inspections, sketches of the various sites should be made as well as an overall sketch or small-scale map showing all of the sites under consideration. If possible, an aerial inspection of the various sites should be made and aerial photographs should be taken.
2. On the final inspection, those sites selected during the preliminary inspection are visited. If federal-aid funds are involved, an FAA airports representative should accompany the inspection group.

Evaluation and Recommendations. The FAA recommends that a final report be prepared including a rough cost estimate for each site. The report should list the sites in order of preference and indicate the advantages and disadvantages of each site. In cases involving federal aid, this report is submitted to the FAA for endorsement, and a site

[1] The airport sponsor is usually a state, city, or other local body, although it may be a private organization or individual.

selecting airports

mutually agreeable to the sponsor and the FAA is chosen. Written endorsement by the FAA does not imply a commitment of federal funds, but it is a necessary step in obtaining funds under the federal-aid airports program.

If no federal funds are requested, the FAA should be contacted for further information regarding the initiation of a request for airspace review.

There are at least 10 factors that should be considered when analyzing a potential airport site:

1. Convenience to users
2. Availability of land and land costs
3. Design and layout of the airport
4. Airspace obstructions
5. Engineering factors
6. Social and environmental factors
7. Availability of utilities
8. Atmospheric conditions
9. Hazards due to birds
10. Coordination with other airports

Convenience to Users. If it is to be successful, an airport must be conveniently located to those who use it. From this viewpoint, the airport ideally would be located near the center of most cities. The obvious problems of air obstructions and land costs rule out this possibility, and most cities have found it necessary to locate the airport several miles from the city center. In major U.S. cities, the average central-city-to-airport distance is about 16 km (10 miles).

Urban sprawl and increasing scarcity and costs of land have resulted in airports being located farther and farther from the city center.[2] At the same time, air speeds have increased, with the result that an increasing percentage of air passengers spend more time in the ground transportation portion of the trip than in the air.[3] It is known that the amount of airport use is very sensitive to the ratio of the ground travel time to the total journey time. As this ratio increases, the air traffic can be expected to decrease significantly, especially if fast alternative ground modes are available, as with the Shinkansen train in Japan and the TGV in France.

In the view of the airport user, travel time is almost invariably a more important measure of convenience than distance. Thus, a relatively remote potential airport site should not be ruled out if it is conveniently located near a major highway or other surface transportation facility.

Availability of Land and Land Costs. Vast acreages are required for major airports, and it is not uncommon for new airports in large cities to require 4050 hectares (10,000 acres) or more. However, the smallest general aviation airports may require less than 40 hectares (100 acres). Table 16-1 gives the minimum recommended land requirements for four

[2]An Arthur D. Little study of 11 cities indicated that in the case of the old airport the average central-city-to-airport distance was 11.3 km (7.0 miles). For these cities, the average distance from the central city to a proposed or new airport was 25.4 km (15.8 miles). The most remote new airport was Dulles International Airport, located 43.5 km (27.0 miles) from downtown Washington, DC [7].

[3]The ratio of ground travel time to total trip time decreased with increases in the total length of the trip. A study of the 50 most heavily traveled city-to-city routes indicated that trips in which the airport-to-airport mileage was 400 km or less (250 miles or less), 51 to 65 percent of the total trip time was spent in ground travel. For trips in which the airport-to-airport distance was 1600 km (1000 miles) or more, this percentage ranged from 22 to 32 percent [7].

Table 16-1 Minimum Land Requirements for Utility Airports

Runway Type	Runway Length ft	m	Landing Area (acres)	Approach Area (acres)	Building Area[a] (acres)	Total Area (acres)
Basic utility stage I	2000	600	23	21	8	52
	3000	900	32	21	8	61
	4000	1200	41	21	8	70
	5000	1500	50	21	8	79
Basic utility stage II	2500	750	27	21	12	60
	3500	1050	36	21	12	69
	4500	1350	45	21	12	78
	5500	1650	54	21	12	87
General utility stage I	3000	900	39	30	24	93
	4000	1200	51	30	24	105
	5000	1500	63	30	24	117
	6000	1800	75	30	24	129
General utility stage II	3500	1050	90	125	24	239
	4500	1350	113	125	24	262
	5500	1650	136	125	24	285
	6500	1950	159	125	24	308

[a]These figures vary due to assumed higher degree of activity at the higher type of airport.

Source: Utility Airports, FAA Advisory Circular AC 150/5300-4B, including Change 8, July 3, 1985. (Now withdrawn.)

classes of utility airports.[4] The amount of land required will depend on the length and number of runways, lateral clearances, and areas required for buildings, aprons, automobile parking and circulation, and so forth.

Since desirable airport site land is also in demand for other purposes, land costs are high, and real estate can be expected to appreciate with the planning and development of a new airport facility. Land costs for large air carrier airports often amount to hundreds of millions of dollars.

It is important that sufficient land be acquired for future expansion. Failure to do so could mean that a convenient and otherwise desirable airport site would have to be abandoned due to limitations in aircraft operations from an unexpandable runway or inadequate space for aircraft or passenger handling.

The requirements for large acreages, the need for convenient location, and the high land costs may lead to novel approaches to the problem of site selection. For example, consideration was given in the late 1970s to building an airport situated approximately 4.6 km (6 miles) offshore on Lake Erie that would serve Cleveland [8]. That proposal was later found to be infeasible. However, during the 1980s and 1990s, several schemes to locate large international airports in the sea have gone ahead in Osaka, Hong Kong, Macao, and Seoul in the Pacific Rim.

Design and Layout of the Airport. In considering alternate potential airport sites, the basic layout and design essentially should be constant. One should avoid making major departures from the desired layout and design to fit a particular site. In this connection, one consideration is especially important. Runways should be oriented so as to take advantage of prevailing winds, and variations in runway alignment from optimum orientation of more than ±10 degrees normally should not be made. (See Section 16-6 for FAA standards on runway orientation.)

[4]The FAA defines a utility airport as one that serves general aviation aircraft of 5670 kg (12,500 lb) or less.

Airspace and Obstruction. To meet essential needs for in-flight safety, two requirements must be met:

1. Adjacent airports must be located so that traffic using one in no way interferes with traffic using the other. An airspace analysis should be made to ensure that this requirement is met. It is desirable that the assistance of the FAA be sought in conducting this analysis, especially when an airport is to be located in a highly developed terminal complex.
2. Physical objects such as towers, poles, buildings, mountain ranges, and so on, must not penetrate navigable airspace. Criteria on "Objects Affecting Navigable Airspace," given in Federal Aviation Regulation Part 77, should be consulted prior to beginning the site selection process. (See Section 16-7.)

Engineering Factors. An airport site should have fairly level topography and be free of mountains, hills, and so on. Further, the terrain should have sufficient slope that adequate drainage can be provided. Areas that require extensive rock excavation should be avoided, as should sites containing peat, muck, and otherwise undesirable foundation materials.

An adequate supply of aggregates and other construction materials should be located within a reasonable distance of the site.

A desirable airport site will be relatively free of timber, although a border of timber along the airport periphery may suppress undesirable noise.

Social and Environmental Factors. One of the most difficult social problems associated with airport location is that of noise. With the advent of the jet, aircraft engine airport noise has worsened and, despite efforts by industry and government groups, the development of a quiet aircraft engine does not seem likely within the near future.

Airports are not good neighbors, and some control in the development of land surrounding an airport should be exercised. In selecting an airport site, proximity to residential areas, schools, and churches should be avoided, and the runways should be oriented so that these land uses do not fall in the immediate approach–departure paths.

As a result of several federal environmental laws passed during the period from 1966 onwards, applicants for federal aid for airport construction must now prepare environmental impact statements for all airport developments that would significantly affect the quality of the environment. References 9 and 10 provide guidance for the preparation of environmental impact statements for U.S. airports. Similar guidelines have been published by the International Civil Aviation Organization for airports throughout the world [11].

Availability of Utilities. With rare exceptions (e.g., the Dulles International Airport, Washington, D.C.), airports must depend upon existing utilities. The site should be accessible to water, electrical service, telephones, gas lines, and so on, and these utilities should be of proper type and size.

Atmospheric Conditions. Peculiar atmospheric conditions such as fog, smoke, snow, or glare may rule out the use of some potential airport sites.

Hazards Due to Birds. Aircraft impact with birds and bird ingestion into turbine engines have caused numerous air disasters. Airports should not be situated near bird habitats or natural preserves and feeding grounds. At certain potential sites, special work such as filling of ponds and closing of dumps may be required to ensure that birds will not present a hazard to aircraft flights.

Coordination with Other Airports. Studies of aviation activity in heavily populated metropolitan areas indicate that more than one major airport will be required in order to meet future air travel needs. Clearly, where two or more large cities are closely spaced (e.g., Seattle and Tacoma, Baltimore and Washington, D.C., etc.), individual airport requirements must be determined in relation to the needs of the entire metropolitan area, and each airport must be considered as a part of a total system.

16-6. RUNWAY ORIENTATION

Because of the obvious advantages of landing and taking off into the wind, runways are oriented in the direction of prevailing winds. Aircraft may not maneuver safely on a runway when the wind contains a large component at right angle to the direction of travel. The point at which this component (called the crosswind) becomes excessive will depend upon the size and operating characteristics of the aircraft. Recommended limiting crosswinds for aircraft operations are 10.5 knots for Airport Reference Codes A-I and B-I, 13 knots for Airport Reference Codes A-II and B-II, 16 knots for Airport Reference Code A-III, B-III, and C-I through D-III, and 20 knots for Airport Reference Codes A-IV through D-VI [12]. The FAA Airport Reference Code system is described in Section 18-3.

According to FAA standards, runways should be oriented so that aircraft may be landed at least 95 percent of the time without exceeding the allowable crosswinds. Where a single runway or set of parallel runways cannot be oriented to provide 95 percent wind coverage, construction of one or more crosswind runways may be necessary.

Wind Rose Method. A graphical procedure utilizing a wind rose may be used to determine the "best" runway orientation insofar as prevailing winds are concerned (See Fig. 16-3.)

For U.S. airports, wind data are usually available from the National Oceanic and Atmospheric Administration, National Climatic Center, Asheville, North Carolina. A record of 10 consecutive years of wind observations should be utilized if available. If suitable weather records are not available accurate wind data for the area should be collected. (Another alternative would be to form a composite wind record from nearby wind-recording stations.) The wind data are arranged according to velocity, direction, and frequency of occurrence as shown by Table 16-2. This table indicates the number of hours wind velocities within a certain range and from a given direction were observed. For example, the table indicates that for the hypothetical site, northerly winds in the 11 to 16-knot range can be expected approximately $(569/87{,}864) \times 100\% = 0.6$ percent of the time.

These data are plotted on a wind rose by placing the percentages in the appropriate segment of the graph. On the wind rose, the circles represent wind velocity in knots, and the radial lines indicate wind direction. The data from Table 16-2 have been plotted properly on Fig. 16-3.

The wind rose procedure makes use of a transparent template on which three parallel lines have been plotted. The middle line represents the runway center line and the distance between it and each of the outside lines is equal to the allowable crosswind component (e.g., 10 kt).

The following steps are necessary to determine the "best" runway orientation and to determine the percentage of time that orientation conforms to the crosswind standards.

1. Place the template on the wind rose so that the middle line passes through the center of the wind rose.
2. Using the center of the wind rose as a pivot, rotate the template until the sum of the percentages between the outside lines is a maximum. When the template strip

19849

95(

956

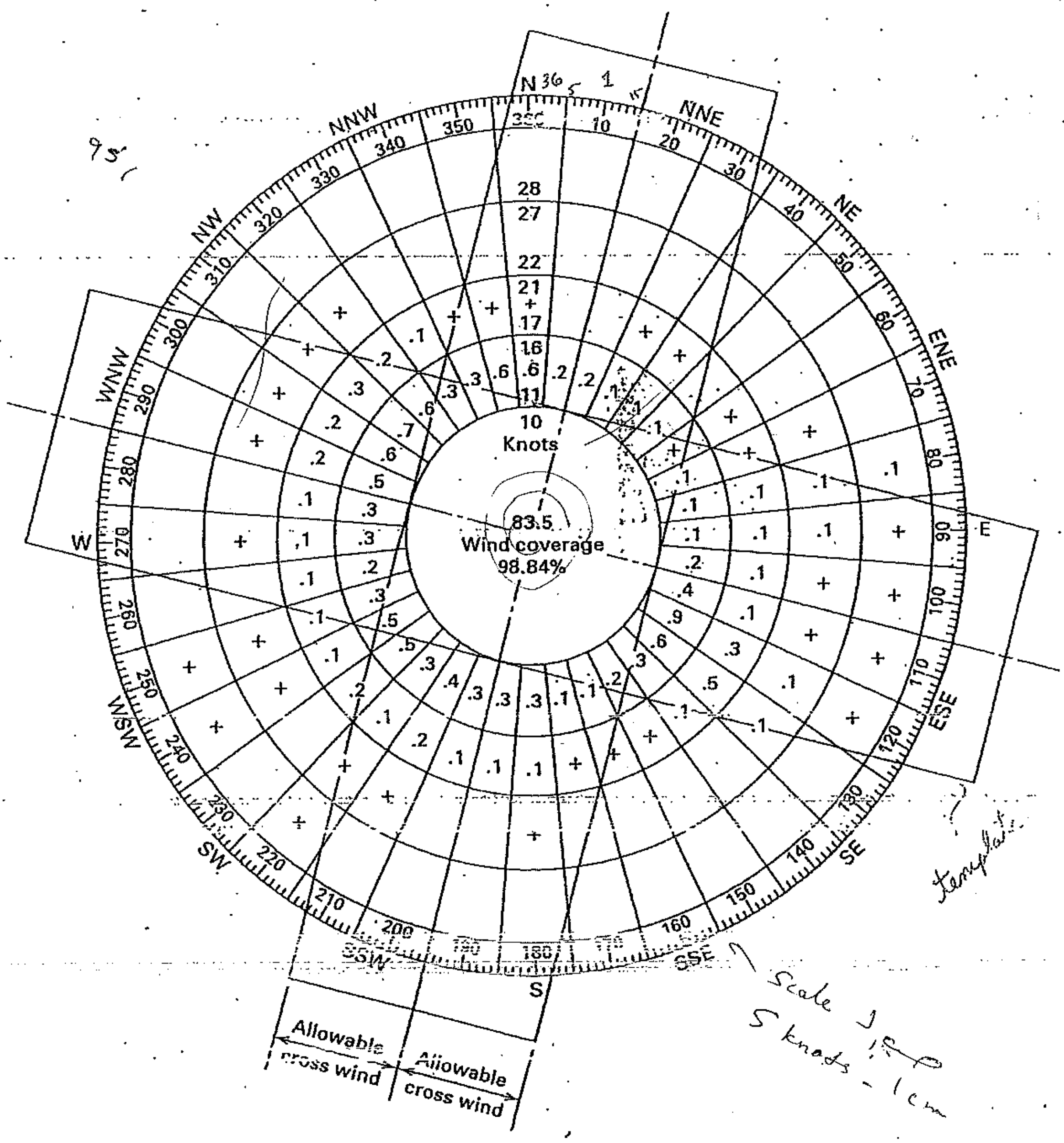


Figure 16-3 A typical wind rose.

covers only a fraction of a segment, a corresponding fractional part of the percentage shown should be used.

3. Read the true bearing for the runway on the outer scale of the wind rose beneath the center line of the template.
4. The sum of percentages between the outside lines indicates the percentage of time that a runway with the proposed orientation will conform with crosswind standards. In the example, the orientations shown for two bi-directional runways give a wind coverage of 98.84 percent.

While the hand method described above is useful in giving designers a "feel" for the wind coverage of individual runways and combinations of runways, computerized methods are available for optimizing wind coverage using wind table input.

It is noted that wind data are gathered and reported with true north as a reference, while runway orientation and numbering are based on the magnetic azimuth. (See Section

Table 16-2 Typical Wind Data

	Hourly Observations of Wind Speed (Knots)									
	0–3	4–6	7–10	11–16	17–21	22–27	28–33	34–40	41 Over	Total
Direction										
1	469	842	568	212	0	0	0	0	0	2091
2	568	1263	820	169	0	0	0	0	0	2820
3	294	775	519	73	9	0	0	0	0	1670
4	317	872	509	62	11	0	0	0	0	1771
5	268	861	437	106	0	0	0	0	0	1672
6	357	534	151	42	8	0	0	0	0	1092
7	369	403	273	84	36	10	0	0	0	1175
8	158	261	138	69	73	52	41	22	0	814
9	167	352	176	128	68	59	21	0	0	971
10	119	303	127	180	98	41	9	0	0	877
11	323	586	268	312	111	23	28	0	0	1651
12	618	1397	624	779	271	69	21	0	0	3779
13	472	1375	674	531	452	67	0	0	0	3571
14	647	1377	574	281	129	0	0	0	0	3008
15	338	1093	348	135	27	0	0	0	0	1941
16	560	1399	523	121	19	0	0	0	0	2622
17	587	883	469	128	12	0	0	0	0	2079
18	1046	1984	1068	297	83	18	0	0	0	4496
19	499	793	586	241	92	0	0	0	0	2211
20	371	946	615	243	64	0	0	0	0	2239
21	340	732	528	323	147	8	0	0	0	2078
22	479	768	603	231	115	38	19	0	0	2253
23	187	1008	915	413	192	0	0	0	0	2715
24	458	943	800	453	96	11	18	0	0	2779
25	351	899	752	297	102	21	9	0	0	2431
26	368	731	379	208	53	0	0	0	0	1739
27	411	748	469	232	118	19	0	0	0	1997
28	191	554	276	287	118	0	0	0	0	1426
29	271	642	548	479	143	17	0	0	0	2100
30	379	873	526	543	208	34	0	0	0	2563
31	299	643	597	618	222	19	0	0	0	2398
32	397	852	521	559	158	23	0	0	0	2510
33	236	721	324	238	48	0	0	0	0	1567
34	280	916	845	307	24	0	0	0	0	2372
35	252	931	918	487	23	0	0	0	0	2611
36	501	1568	1381	569	27	0	0	0	0	4046
0	7729	0	0	0	0	0	0	0	0	7729
TOTAL:	21676	31828	19849	10437	3357	529	166	22	0	87864

Source: Airport Design, FAA Advisory Circular 150/5300-13, including Changes 1–4, Federal Aviation Administration, Washington, DC, September 29, 1989.

$\frac{21676}{87864} \times 100$

18-12.) The true azimuth obtained from the wind rose analysis should be changed to a magnetic azimuth by taking into account the magnetic variation[5] for the airport location. An easterly variation is subtracted from the true azimuth, and a westerly variation is added to the true azimuth.

[5]The magnetic variation can be obtained from aeronautical charts.

16-7. OBJECTS AFFECTING NAVIGABLE AIRSPACE

Part 77 of the Federal Aviation Regulations (FAR) [13] establishes standards for determining obstructions in navigable airspace, sets forth the notice requirements of certain proposed construction or alteration, provides for aeronautical safety studies of obstructions, and provides for public hearings on the hazardous effect of proposed construction or alteration. The FAA regulations describe imaginary surfaces that define airspace that should be free of objects hazardous to air navigation. If an obstacle (for example, a building, radio tower, or mountain range) penetrates any of these surfaces, it is classified as an obstruction, requiring further study in collaboration with the FAA to determine if it would constitute a hazard to air navigation.

Notification Requirements. FAR Part 77 [13] sets forth a procedure by which the FAA must be notified of proposed construction or alteration activities that might affect navigable airspace. Depending on the effect that the obstacle might have on air navigation, the FAA is authorized to convene formal hearings and, on the basis of evidence presented at such hearings, to determine appropriate measures to be applied for continued safety of air navigation. Such measures include requiring that an obstacle be removed or be marked and lighted properly [14] or that the plans for construction or alteration be modified or canceled.

Generally speaking, an airport sponsor must notify the FAA of any construction or alteration of any airport that is available for public use and is in the Airport Directory of the current Aeronautical Information Manual [15]. Similarly, the FAA must be notified of new civilian airports that are under construction and are expected to become available for public use and also must be notified of changes to airports operated by one of the military services.

In addition, an airport sponsor must notify the FAA of any construction alteration:

- Of more than 61 m (200 ft) in height above the ground level at the site
- That penetrates the surfaces shown in Fig. 16-4
- Within an instrument approach area where available information indicates that an obstacle might exceed a standard of Subpart C of FAR Part 77, which gives standards for evaluating obstructions to air navigation

The height of highways, railroads, and waterways must be adjusted upward when determining if the notification requirements are met. The adjustments are:

For Interstate highways	5.2 m (17 ft)
For other public roadways	4.6 m (15 ft)
For railroads	7.0 m (23 ft)

For private roads and waterways, the height usually must be adjusted by the height of the highest vehicle that will use the traverse way.

Obstruction Standards. According to Subpart C of FAR Part 77, an object is an obstruction to air navigation if it is of greater height than any of the following heights of surfaces:

1. A height of 152 m (500 ft) above ground level at the site of the object.
2. A height that is 61 m (200 ft) above ground level or above the established airport elevation, whichever is higher, within 3 nautical miles of the established reference point of an airport excluding heliports, with its longest runway more than 475 m (3200 ft) in actual length, and that height increases in the proportion of 30.5 m (100 ft) for each additional nautical mile of distance from the airport up to a maximum of 152 m (500 ft). (See Fig. 16-5.)

X_1

X_2

$2X_1 + 3X_2$

$X_1 + \frac{X_2}{2} \le 10$

$2X_1 + X_2 \le 80$

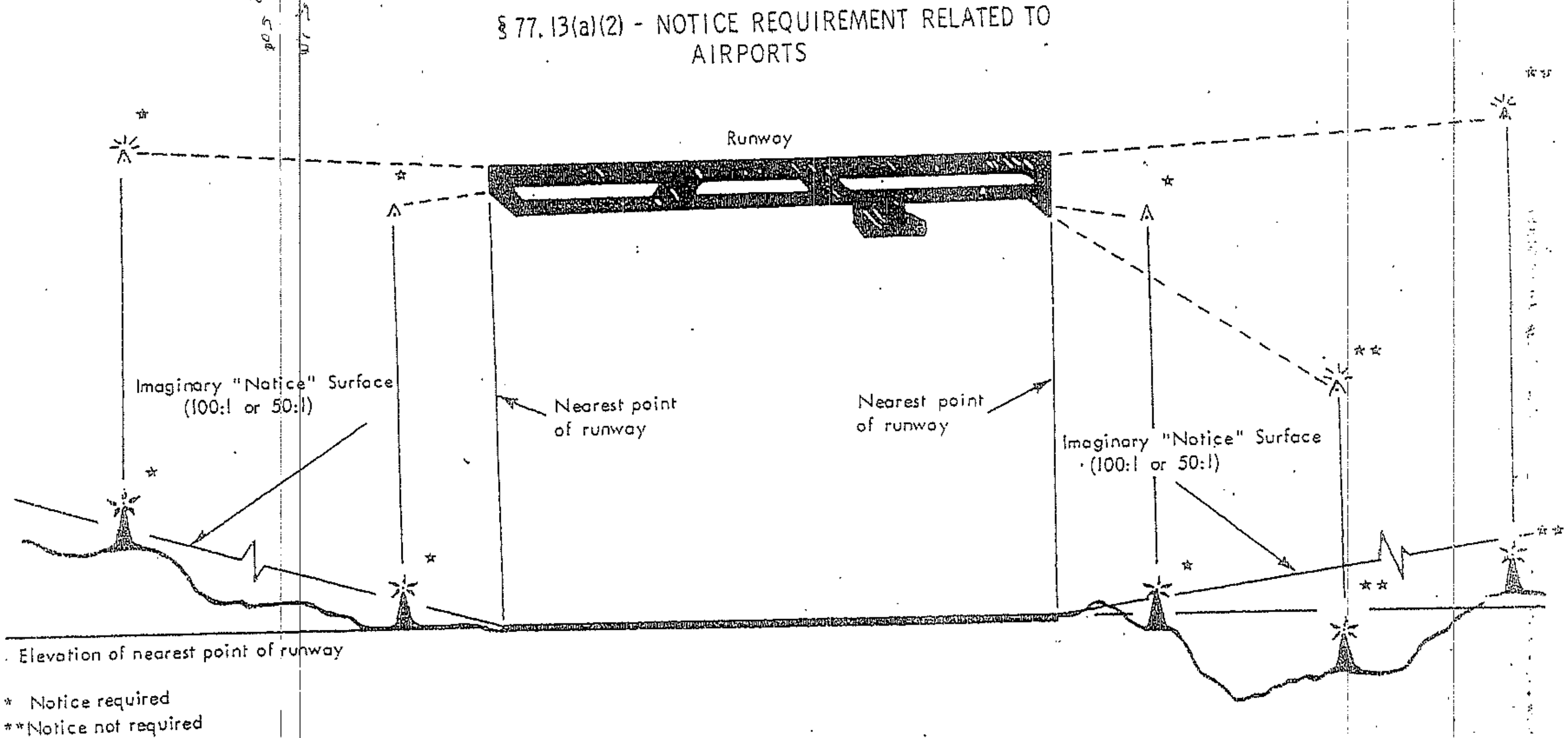
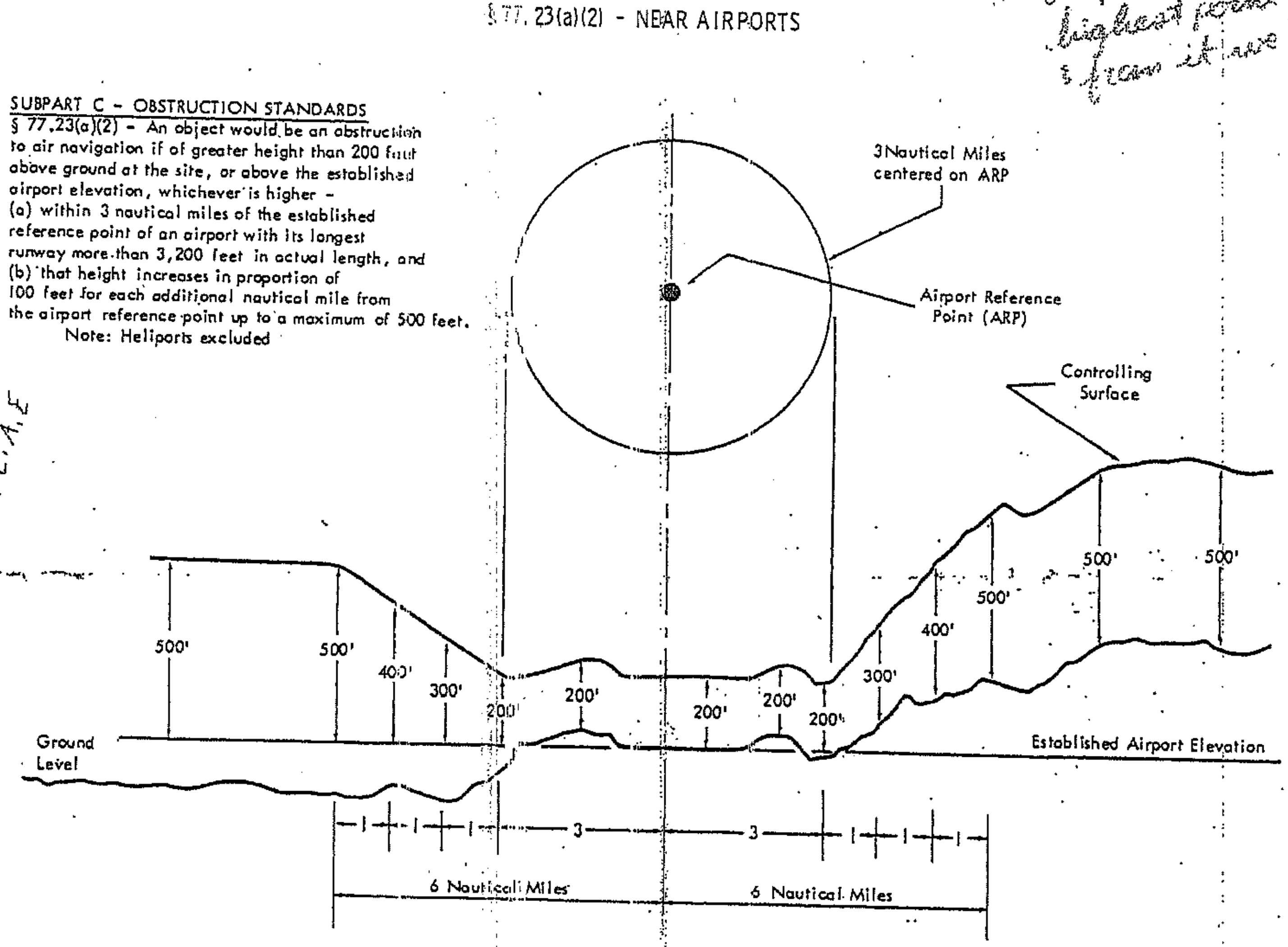


Figure 16-4 FAA notice requirements for airport construction or alteration. (Source: *Federal Aviation Regulations, Part 77,* 1975.)

Airport reference pt = highest point on runway & from it we take heights



we measure from the higher G.L or E.A.E

IHP

Figure 16-5 Obstruction standards in the vicinity of airports. (Source: *Federal Aviation Regulations, Part 77,* 1975.)

3. A height within a terminal obstacle clearance area, including an initial approach segment, a departure area, and a circling approach area, that would result in the vertical distance between any point on the object and an established minimum instrument flight altitude within that area or segment to be less than the required obstacle clearance.
4. A height within an en-route obstacle clearance area, including turn and termination areas, of a federal airway or approved off-airway route, that would increase the minimum obstacle clearance altitude.
5. The surface of a takeoff and landing area of an airport or any imaginary surface described in the following paragraphs. (See Fig. 16-6.)

Civil Airport Imaginary Surfaces. The FAA has established imaginary surfaces as a means of checking the effect of objects in the vicinity of airports and approaches. These surfaces are illustrated in Fig. 16-6. Their dimensions are described in the following paragraphs, taken from reference 13, with but minor editorial changes.

The size of each such imaginary surface is based on the category of each runway according to the type of approach available or planned for that runway. The slope and dimensions of the approach surface applied to each end of a runway are determined by the most precise approach existing or planned for that runway end.

(a) Horizontal surface—a horizontal plane 45.7 m (150 ft) above the established airport elevation,[6] the perimeter of which is constructed by swinging arcs of specified radii from the center of each end of the primary surface of each runway of each airport and connecting the adjacent arcs by lines tangent to those arcs. The radius of each arc is:

(1) 1524 m (5000 ft) for all runways designated as utility or visual;

(2) 3048 m (10,000 ft) for all other runways.

The radius of the arc specified for each end of a runway will have the same arithmetical value. That value will be the highest determined for either end of the runway. When a 1524 m (5000-ft) arc is encompassed by tangents connecting two adjacent 3048-m (10,000-ft) arcs, the 1524-m (5000-ft) arc shall be disregarded on the construction of the perimeter of the horizontal surface.

(b) Conical surface—a surface extending outward and upward from the periphery of the horizontal surface at a slope of 20 to 1 for a horizontal distance of 1219 m (4000 ft).

(c) Primary surface—a surface longitudinally centered on a runway. When the runway has a specially prepared hard surface, the primary surface extends 61 m (200 ft) beyond each end of that runway; but when the runway has no specially prepared hard surface, or planned hard surface, the primary surface ends at each end of that runway. The elevation of any point on the primary surface is the same as the elevation of the nearest point on the runway centerline. The width of a primary surface is:

Length

L = RW length + 200' from each side of RW

(1) 76 m (250 ft) for utility runways having only visual approaches.

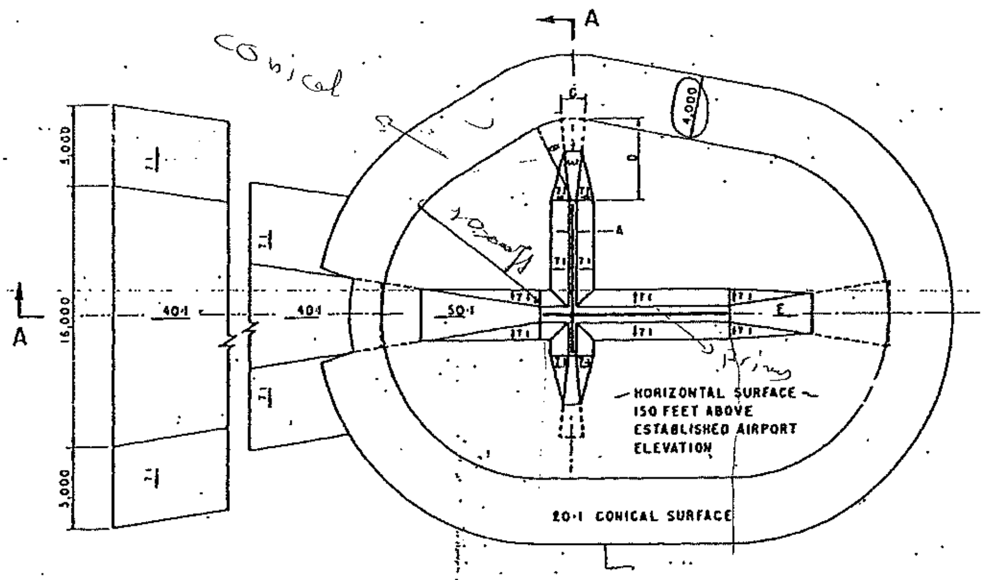
(2) 152 m (500 ft) for utility runways having nonprecision instrument approaches.

(3) For other than utility runways the width is:

(i) 152 m (500 ft) for visual runways having only visual approaches.

(ii) 152 m (500 ft) for nonprecision instrument runways having visibility minimums greater than three-fourths statute mile.

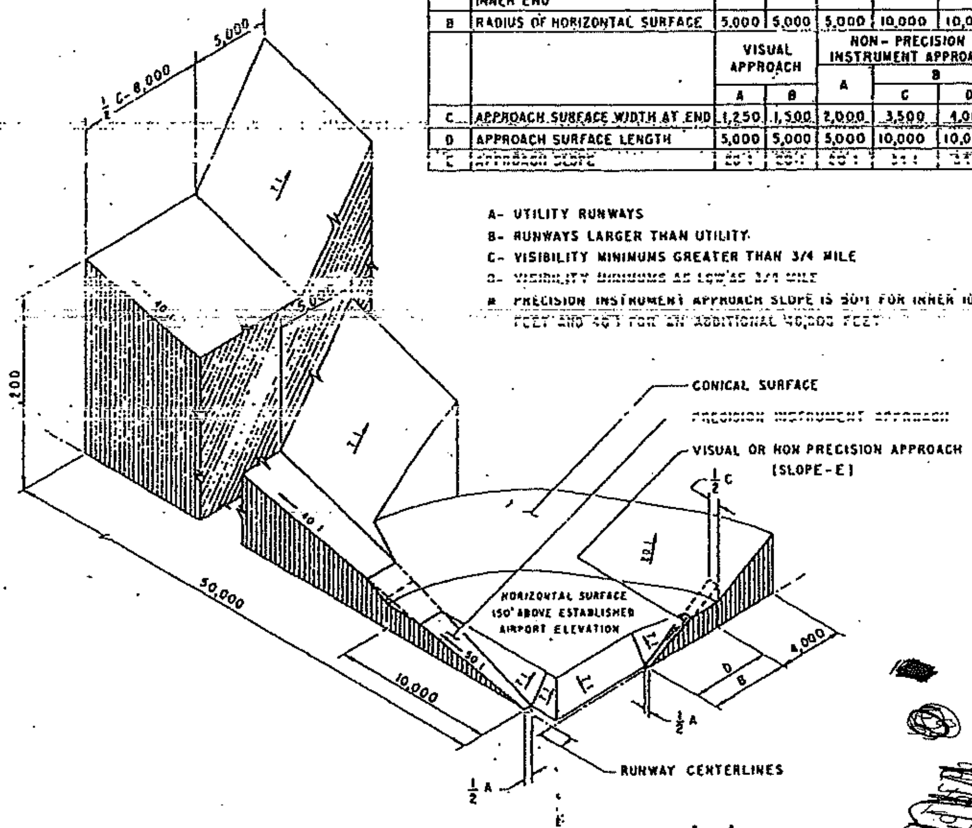
[6]The established airport elevation is usually taken to be the highest elevation on the runway system.



DIM	ITEM	DIMENSIONAL STANDARDS (FEET)					
		VISUAL RUNWAY		NON-PRECISION INSTRUMENT RUNWAY			PRECISION INSTRUMENT RUNWAY
					B		
		A	B	A	C	D	
A	WIDTH OF PRIMARY SURFACE AND APPROACH SURFACE WIDTH AT INNER END	250	500	500	500	1,000	1,000
B	RADIUS OF HORIZONTAL SURFACE	5,000	5,000	5,000	10,000	10,000	10,000
		VISUAL APPROACH		NON-PRECISION INSTRUMENT APPROACH			PRECISION INSTRUMENT APPROACH
					B		
		A	B	A	C	D	
C	APPROACH SURFACE WIDTH AT END	1,250	1,500	2,000	3,500	4,000	16,000
D	APPROACH SURFACE LENGTH	5,000	5,000	5,000	10,000	10,000	*
E	APPROACH SLOPE	20:1	20:1	20:1	34:1	34:1	*

A- UTILITY RUNWAYS
B- RUNWAYS LARGER THAN UTILITY
C- VISIBILITY MINIMUMS GREATER THAN 3/4 MILE
D- VISIBILITY MINIMUMS AS LOW AS 3/4 MILE
* PRECISION INSTRUMENT APPROACH SLOPE IS 50:1 FOR INNER 10,000 FEET AND 40:1 FOR AN ADDITIONAL 40,000 FEET

our concern



ISOMETRIC VIEW OF SECTION A-A

§ 77.25 CIVIL AIRPORT IMAGINARY SURFACES

Figure 16-6 Civil airport imaginary surfaces. (Source: *Federal Aviation Regulations, Part 77*, 1975.)

Elevation = 0

(iii) 305 m (1000 ft) for a nonprecision instrument runway approach having a nonprecision instrument with visibility minimums as low as three-fourths of a statute mile, and for precision instrument runways.

The width of the primary surface of a runway will be that width prescribed in this section for the most precise approach existing or planned for either end of that runway.

(d) Approach surface—a surface longitudinally centered on the extended runway center line and extending outward and upward from each end of the primary surface. An approach surface is applied to each end of each runway based upon the type of approach available or planned for that runway end.

(1) The inner edge of the approach surface is the same width as the primary surface and it expands uniformly to a width of:

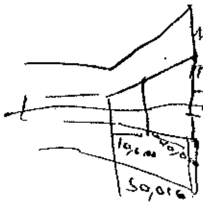
(i) 381 m (1250 ft) for that end of a utility runway with only visual approaches;

(ii) 457 m (1500 ft) for that end of a runway other than a utility runway with only visual approaches;

(iii) 610 m (2000 ft) for that end of a utility runway with a nonprecision instrument approach;

(iv) 1067 m (3500 ft) for that end of a nonprecision instrument runway other than utility, having visibility minimums greater than three-fourths of a statute mile;

(v) 1219 m (4000 ft) for that end of a nonprecision instrument runway, other than utility, having a nonprecision instrument approach with visibility minimums as low as three-fourths statute mile; and



(vi) 4877 m (16,000 ft) for precision instrument runways.

(2) The approach surface extends for a horizontal distance of:

(i) 1524 m (5000 ft) at a slope of 20 to 1 for all utility and visual runways;

(ii) 3048 m (10,000 ft) at a slope of 34 to 1 for all nonprecision instrument runways other than utility; and,

(iii) 3048 m (10,000 ft) at a slope of 50 to 1 with an additional 12,192 m (40,000 ft) at a slope of 40 to 1 for all precision instrument runways.

(3) The outer width of an approach surface to an end of a runway will be that width prescribed in this subsection for the most precise approach existing or planned for that runway end.

(e) Transitional surface—these surfaces extend outward and upward at right angles to the runway centerline and the runway centerline extended at a slope of 7 to 1 from the sides of the primary surface and from the sides of the approach surfaces. Transitional surfaces for those portions of the precision approach surface that project through and beyond the limits of the conical surface, extend a distance of 1524 m (5000 ft) measured horizontally from the edge of the approach surface and at right angles to the runway centerline.

16-8.* RUNWAY CAPACITY

Runway capacity, which is normally the determining element of airport capacity, refers to the ability of a runway system to accommodate aircraft operations (i.e., landings and take-offs). It is expressed in operations per unit time, typically operations per hour or operations per year. Airport capacity research [16] has provided procedures for the determination of the *ultimate* or *saturation* airfield capacities. Saturation capacity is based on the assumption of a continuous backlog of aircraft waiting to take off and land. It does not represent a desirable mode of operation but can be useful in comparing alternative layouts and designs.

Previously, the FAA employed a concept of practical capacity, an empirical-based measure that corresponds to a specified "reasonable" or "tolerable" delay. The preferred measure, and that employed in this chapter, is the ultimate or saturation capacity, the maximum number of aircraft operations that can be handled during a given period under conditions of continuous demand.

Factors That Affect Capacity. The capacity of a runway system is limited by a number of factors. They are summarized as follows:

(1)

- ***Weather and air traffic control conditions*** Runways may be operated either under IFR or VFR conditions. The separations required under conditions of instrument operation are significantly larger than under visual operations. It will be recalled from Section 5-27 that VFR operations are made in good weather conditions and the aircraft is operated by visual reference to the ground. The IFR operations are made in periods of inclement weather and poor visibility, and under these conditions, positive traffic control is maintained by radar and other electronic devices. Runway capacity under IFR conditions is normally less than under VFR conditions.

IFR has less capacity than VFR

- ***Number and configuration of runways*** As the number of runways increases, the capacity of the system tends to increase, provided that the runways can be operated either entirely or partially independently. This factor is discussed in Section 16-9.

nonlinear

- ***Fleet mix*** The higher the percentage of large and heavy aircraft, the lower is the capacity that can be achieved due to larger separations required due to wake turbulence.
- ***Arrival/departure ratio*** Because arriving aircraft tend to follow a single approach track for greater distances and because approach speeds are slower than take-off speeds, the higher the arrival/departure ratio, the lower the capacity of the runway system.
- ***Touch-and-go operations*** The rapid juxtaposition of the arrival and departure operations in touch-and-go training operations leads to higher capacities as the percentage of touch-and-go operations increases.
- ***Number and location of runway exits*** The number of runway exits tends to decrease runway occupancy time of arriving aircraft, provided that these exits are independent and are located in that portion of the runway where they can contribute to making exits more efficient.

Procedures for Estimating Runway Capacities A discussion of detailed procedures for estimating capacities is beyond the scope of this book. Such matters are treated extensively in other publications [17, 18].

The FAA has published approximate hourly and annual service volumes (ASVs) for various runway configurations. Some relatively simple runway configurations are shown in Table 16-3 along with hourly capacities and ASVs.

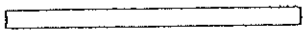
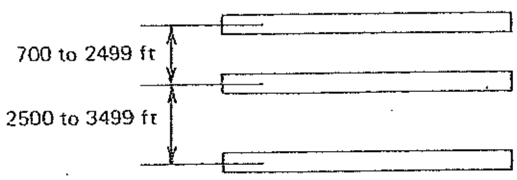
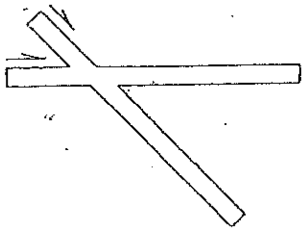
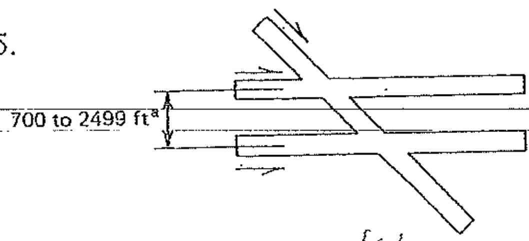
In the table, the aircraft mix is expressed in terms of four aircraft "classes":

Class A: small single-engine aircraft, 12,500 lb or less
Class B: small multiengine aircraft, 12,500 lb or less and Learjets
Class C: large aircraft, 12,500 lb and up to 300,000 lb
Class D: heavy aircraft, more than 300,000 lb

The "mix index" is determined by the percentages of aircraft in classes C and D.

$$\text{Mix index} = (\text{percent aircraft in class C}) + 3(\text{percent aircraft in class D})$$

Table 16-3 Airport Capacities for Long-Range Planning Purpose

Runway Use Configuration	Mix Index (Percent) (C + 3D)	Hourly (hr) Capacity (Operations) ~~VRF~~ VFR	IFR	Annual Service Volume (Operations/yr)
1. 	0–20	98	59	230,000
	21–50	74	57	195,000
	51–80	63	56	205,000
	81–120	55	53	210,000
	121–180 could be higher	51	50	240,000
2. 	0–20	197	59	355,000
	21–50	145	57	275,000
	51–80	121	56	260,000
	81–120	105	59	285,000
	121–180	94	60	340,000
3. 	0–20	295	62	385,000
	21–50	219	63	310,000
	51–80	184	65	290,000
	81–120	161	70	315,000
	121–180	146	75	385,000
4. 	0–20	98	59	230,000
	21–50	77	57	200,000
	51–80	77	56	215,000
	81–120	76	59	225,000
	121–180	72	60	265,000
5. 	0–20	197	59	355,000
	21–50	145	57	275,000
	51–80	121	56	260,000
	81–120	105	59	285,000
	121–180	94	60	340,000

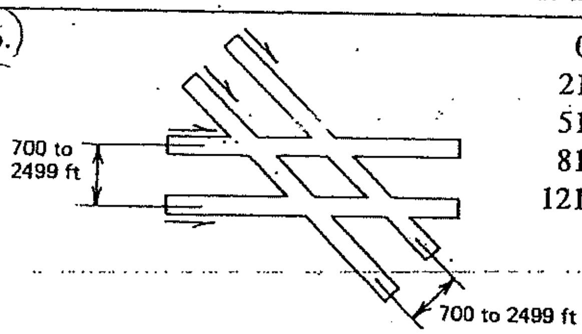
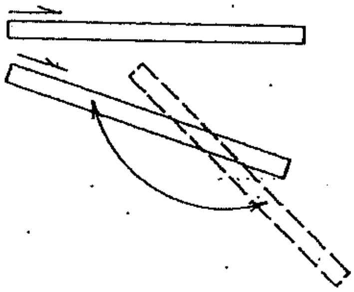
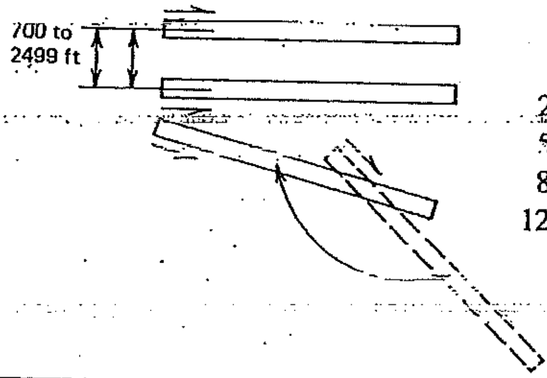
Continued on following page

with angles

A 30 → 10%
B 50 ~ 16.5%
C 200 ~ 66.95%
D 20 ~ 6.75%
Σ 300

mix index = % + 3(%D)
= 87

Table 16-3 *Continued*

6.		0–20	197	59	355,000
		21–50	147	57	275,000
		51–80	145	56	270,000
		81–120	138	59	295,000
		121–180	125	60	350,000
7.		0–20	150	59	270,000
		21–50	108	57	225,000
		51–80	85	56	220,000
		81–120	77	59	225,000
		121–180	73	60	265,000
8.		0–20	295	59	385,000
		21–50	210	57	305,000
		51–80	164	56	275,000
		81–120	146	59	300,000
		121–180	129	60	355,000

[a]Staggered threshold adjustments may apply

Source: Airport Capacity and Delay, AC 150/5060.5, Federal Aviation Administration, Washington, DC, Sept. 23, 1983.

* It should be noted that the data given in Table 16-3 are based on average traffic and airport conditions. Procedures for making adjustments to the capacity values for local variations in traffic, airport layout, and weather are given by reference 17. The capacities shown in Table 16-3 are recommended only for long-range planning purposes.

16-9. RUNWAY CONFIGURATION

Inherent in the layout of an airport is the need to arrange a given runway efficiently in relation to other runways and service facilities such as the terminal building, aprons, hangars, and other airport buildings. As was indicated earlier, runway configuration is a principal factor affecting runway capacity. Practically speaking, it is the only factor an airport planner can change and bring about an increase in capacity to serve future demand.

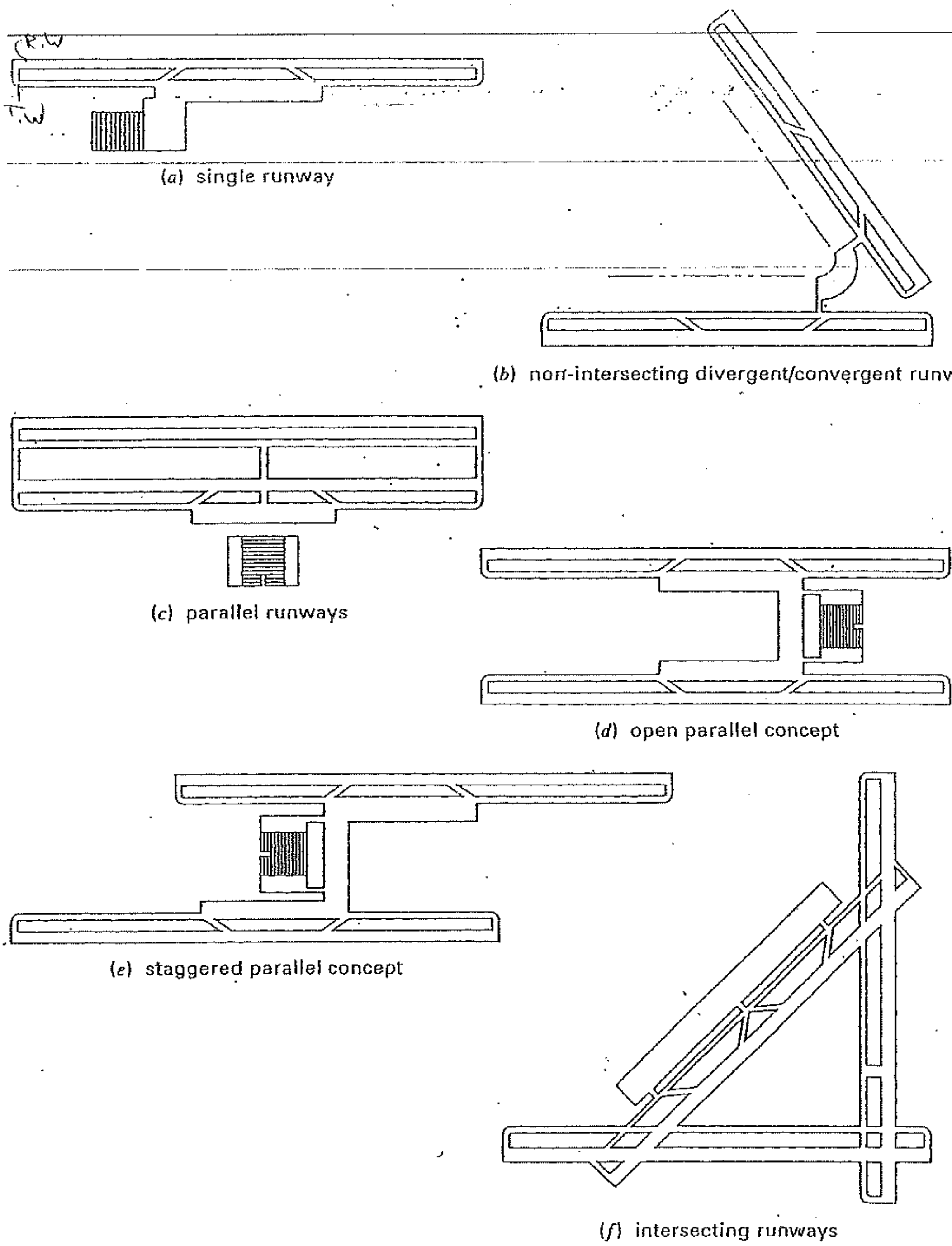


Figure 16-7 Typical airport configurations. (*a*) Single runway. (*b*) Nonintersecting divergent/convergent runways. (*c*) Parallel runways. (*d*) Open parallel runways (*e*) Staggered parallel concept. (*f*) Intersecting runways.

Typical runway configurations are shown schematically in Fig. 16-7 and the first column of Table 16-3.

The simplest runway configuration is a single-runway system, which has an hourly capacity of 51 to 98 operations in VFR conditions and 50 to 59 operations under IFR conditions. Frequently, a second runway is added to take advantage of a wider range of wind direction. This system has a higher capacity than a single-runway system, provided winds are not strong. In conditions of high winds and poor visibility, this system operates as a single runway.

An airport's runway capacity can be increased by adding a second parallel runway. With a separation of at least 213 m (700 ft), simultaneous landings in the same direction[7] may be made on the parallel runways under VFR conditions with a resulting capacity of 94 to 197 operations per hour. With a runway separation of at least 1311 m (4300 feet), simultaneous operation may be made under IFR conditions, and the IFR capacity of this system is 99 to 119 operations per hour.

The parallel runway system shown in Fig. 16-7*c* has the disadvantage of requiring aircraft using the outboard runway to taxi across the runway adjacent to the terminal area. This disadvantage may be overcome by placing the terminal facilities between the two runways, as shown in Fig. 16-7*d*.

In a location where there are prevailing winds from one direction a large percentage of the time, the parallel runways may be staggered or placed in tandem, as shown in Fig. 16-7*e*. This makes it possible to reduce taxiway distances by using one runway exclusively for take-off operations and the other runway for landings. This configuration, however, requires a great deal of land.

Large airports may require three or more runways. The best configuration for a multiple runway system will depend on the minimum spacing required for safety, prevailing wind directions, topographic features of the airport site, shape and amount of available space, and the space requirements for aprons, the terminal, and other buildings.

PROBLEMS

1. Using the wind data given here, construct a wind rose for an airport with Airport Reference Code BII and indicate what would be the best orientation for a runway based on these prevailing winds. If available, check your results with the latest version of the FAA's computer program, "Airport Design for Microcomputers" [12].

	Percentage of Winds		
Wind Direction	17–21 kt	22–27 kt	>27 kt
1	0.2		
2	0.4	0.1	
3	0.1		
4	0.5	0.1	
5	0.2		0.1
6	0.3		
7	0.1	0.1	
8	0.9	0.1	0.1
9	0.8	0.1	
10	0.8	0.1	
11	0.9	0.2	0.1
12	1.3	0.2	0.1
13	1.1	0.3	0.1
14	1.0	0.1	
15	0.9		<0.1

[7]Simultaneous VFR operations in opposite directions on parallel runways require 427-m (1400-ft) distance between runway centerlines during daylight hours and 853-m (2800-ft) distance during periods of darkness [12].

Wind Direction	Percentage of Winds 17–21 kt	22–27 kt	>27 kt
16	0.9		
17	0.7	0.1	<0.1
18	0.8		
19	0.5		
20	0.2		
21	0.1	0.1	
22	0.1		
23	0.2		
24	0.7		
25	0.6		
26	0.5	0.1	
27	0.4	0.1	
28	0.6	0.2	
29	0.5	0.1	0.1
30	0.8	0.2	0.1
31	0.4	0.1	0.1
32	1.1	0.2	
33	0.5	0.1	
34	0.3	0.1	0.1
35	0.4	0.1	
36	0.2	0.1	
	20.0	3.0	1.0

Percent of winds 0–10 kt = 60.4
Percent of winds 11–16 kt = 15.6

2. For a nearby airport obtain wind data similar to those given in problem 1. Construct a wind rose and determine optimum runway orientation based on prevailing winds. How does this orientation compare with the existing airport's runway(s)?
3. A utility airport is being planned to serve a city of 35,000 people. The visual runway is to be 3600 ft long. Indicate whether or not the following objects will be considered obstructions to air navigation by the FAA. (HINT: 1 nautical mile = 6076 ft.)
   a. A 220-ft radio tower that is not in the landing approach, located 3.1 miles from the airport reference point. The ground elevation at the tower is 25 ft higher than the established airport elevation.
   b. A planned 75-ft-high office building within the landing approach ½ mile from the end of the runway.
4. A new air carrier airport is being planned to serve a city of 300,000 people. The airport will have dual 8000-ft precision instrument runways. Indicate whether or not the following objects constitute an obstruction to air navigation. The established airport elevation is 1220 ft.
   a. A railroad within the approach path located 1.2 miles from the end of the runway with an elevation of 1310 ft. Assume the elevation of the end of the runway is 1220 ft.
   b. A 298-ft radio tower not in the approach path located 3.5 nm from the airport reference point. The ground elevation at the tower is 1275 ft.

5. Estimate the capacity of dual close parallel runways that are to serve the traffic mix shown here under VFR and IFR conditions.

20 percent	Four-engine jet aircraft
36 percent	Two-engine jet and 4-engine piston aircraft
22 percent	Executive jet aircraft
22 percent	Single-engine piston aircraft

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Chapter 17

The Airport Passenger Terminal Area

Typical peak-hr pass vs hourly oper 538

Passenger Term Space Standards

peak hr vs. annual pass. 539

17-1. INTRODUCTION

This chapter will present planning procedures and design criteria for the airport passenger terminal area. For purposes of this chapter, the airport passenger terminal area generally consists of that portion of the airport other than the landing and take-off areas. It includes the automobile parking lots, aircraft parking aprons, passenger terminal building, and facilities for interterminal and intraterminal transportation.

The importance of a well-conceived airport terminal area design can be seen by considering the numerous and varied component movements that a typical airline passenger makes (see Fig. 17-1). A passenger leaves his or her origin and travels to the airport by automobile or one of a variety of public travel modes. From the automobile parking lot or vehicle-unloading platform, the passenger and the baggage move to the ticket and passenger service counter by walking, by moving sidewalks, or by other means. From the gate-loading position, passengers usually walk the short distance to the plane. Additional travel time is involved as the aircraft taxis to the runway holding apron where it waits for control tower clearance to take off. This procedure is essentially reversed at the destination end of the trip. For airline passengers who make en-route stops or transfers, the movements are more numerous and complex.

Each component movement in a typical trip involves possibilities of congestion and delay. It follows that each service facility within the terminal area must be planned carefully and designed to accommodate peak-hour traffic volumes if unacceptable delays are to be avoided. It is also apparent that an integrated layout and design of the airport terminal area is required to provide a smooth uninterrupted flow of people, baggage, and freight. This design must be sufficiently flexible to allow for orderly expansion of service areas without prohibitive costs.

Terminal facilities vary widely in size, design, and layout depending primarily upon the airport type and size and the volume and nature of air traffic.

These facilities at a utility airport, for example, may consist only of two small buildings: (1) a fixed-based operators' (FBO) building that provides space for commercial activities such as aircraft maintenance and repair, air charter, and so on, and (2) an administration building to accommodate pilots, passengers, and visitors and to house the airport manager's office. At the other end of the spectrum there are cities making plans for airport facilities to accommodate up to 100 million annual passengers by providing elaborate terminal complexes with as many as 100 wide-bodied parking positions for commercial jet aircraft and more than that number for conventional jets.

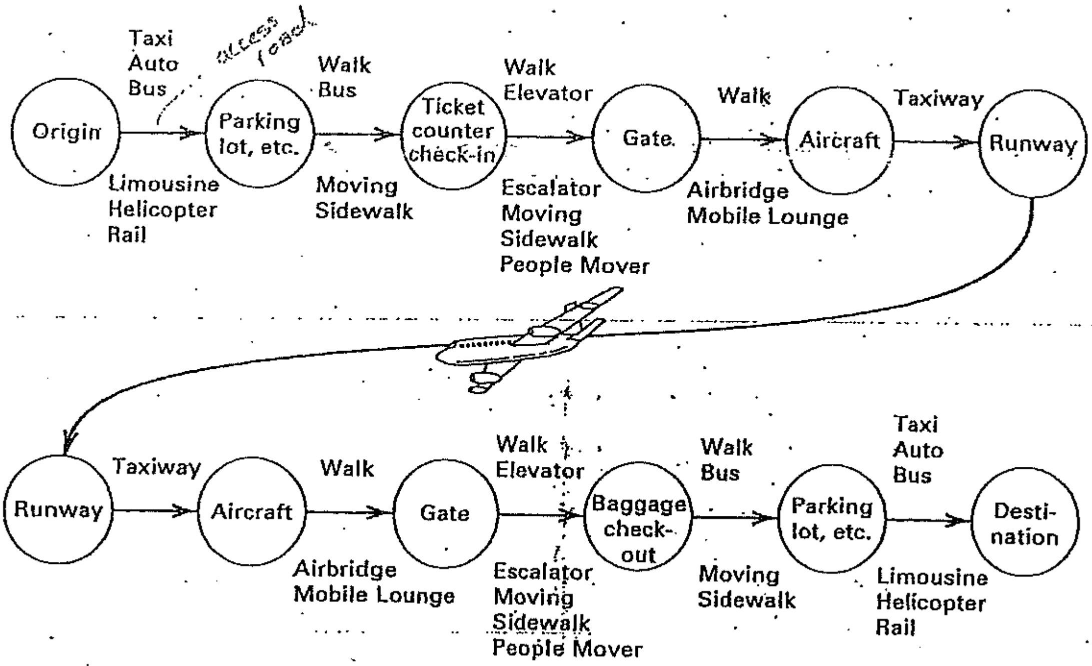


Figure 17-1 A typical air trip.

The primary focus of this chapter will be on medium and large airports (see Table 1-4). ~~Additional information for planning and development of utility airports and nonhub airports is given by references 1 and 2, respectively.~~

17-2 THE AIR TERMINAL PLANNING AND DESIGN PROCESS

The design process for a passenger terminal at a commercial carrier airport is a complex one involving at least four organizations:

The Airport Owner. Commercial carrier airports typically are owned by a municipality or airport authority. The owner is particularly concerned with the financing of the terminal and its operation when completed. By the late 1980s, the privatization of airports had become rather commonplace in various countries in the world, although not in the United States. There are a number of airports, however, such as Indianapolis and Pittsburgh, where the government owners have contracted out the management of the terminals or the whole airport. In such cases, these private management companies become involved in the design process.

The Federal Government. Prior to 1961, the federal government took an active interest in the planning and design of terminal buildings and furnished federal aid for terminal construction. Between 1961 and 1976, when significant amendments were made to the Airport and Airways Development Act of 1970, terminal buildings were no longer eligible for federal aid. In 1976, Public Law 94-353 permitted the use of federal funds for non-revenue-producing areas of the passenger terminal. The federal government also planned for its operational role within the terminal area where immigration, customs, and health inspections may be necessary and where air traffic control facilities are integrated into the terminal design.

The Airlines. As prime tenants, and indeed sometimes the owners of terminal facilities, airlines exert an important influence on the design features of the terminal building and its surroundings. Each airline provides the following estimates of its own terminal needs, usually in stages of 5, 10, 15, and 20 years:

1. Estimates of aircraft movements
2. Estimates of enplaning, deplaning, and interlining passenger traffic
3. Space requirements for ticket counters, baggage handling, services, operations, maintenance, and supply areas
4. Number, type, and size of aircraft parking positions
5. Special requirements such as telecommunications, flight information systems, and passenger transport systems
6. Building and space requirements for air freight
7. Maintenance facilities
8. Fixed ramp facilities such as air conditioning and electric power for aircraft and air bridges
9. Mobile ramp service requirements such as sanitary services, catering, startup, towing vehicles, deicing, water and aircraft steps, and apron transfer vehicles
10. Fuel requirements
11. Automobile parking requirements for employees

Concessionaires. The airport terminal building is a commercial venture of considerable magnitude where consumer services and rentals often produce over half of the terminal revenues. Although most of the concessions are leased after the building is designed, design information on the size and location of restaurants, shops, car rental, and other tenants is required.

Consulting architectural and engineering firms usually are employed by the airport owner to develop a terminal design to satisfy the varied requirements and wishes of the airport management, the federal government, the airlines, and other tenants of the airport.

17-3. AIR TERMINAL LAYOUT CONCEPTS

In the planning of the air terminal the designer weighs a number of objectives: (1) to provide a high level of service to passengers at an acceptable cost in the processing, waiting, and circulation areas by providing adequate space and minimizing the difficulties of transfer movements; (2) to provide a design with a high level of flexibility that can cope with changes in air vehicle technology and changes of passenger traffic levels; (3) to reduce both internal and access walking distances for passengers and taxiing requirements for aircraft; (4) to provide a terminal layout capable of generating potential terminal revenues; and (5) to provide an acceptable working environment for airport and airline staff.

Increasing space needs for both automobiles and aircraft have taxed the designer's ingenuity in recent years. This has been especially true, of course, at large airports. The predominant preference of passengers, visitors, and well-wishers to travel to the airport by automobile largely explains the growing demand for automobile parking spaces. The use of larger aircraft has complicated the designer's task in at least three ways. First, the parking and service area required for these aircraft has increased, making it more difficult to park large numbers of these aircraft within a reasonable walking distance from the processing areas of the terminal building. Second, the sudden deposition of batches of several hundred passengers results in surges of terminal passenger traffic, and space and logistic needs for these peak flows must be provided. Finally, the larger aircraft, with higher tails and greater floor heights, require higher vertical clearances and more flexibility in the heights of loading platforms and bridges.

17-4. AIR TERMINAL LAYOUT SCHEMES

To satisfy the various design objectives, various physical layout schemes have evolved over the years. Six basic terminal layouts are shown in Fig. 17-2. The oldest and simplest concept is the *frontal system*, shown in Fig. 17-2*a*, in which the aircraft park parallel to the terminal building. This system is adequate for small airports where the number of air-

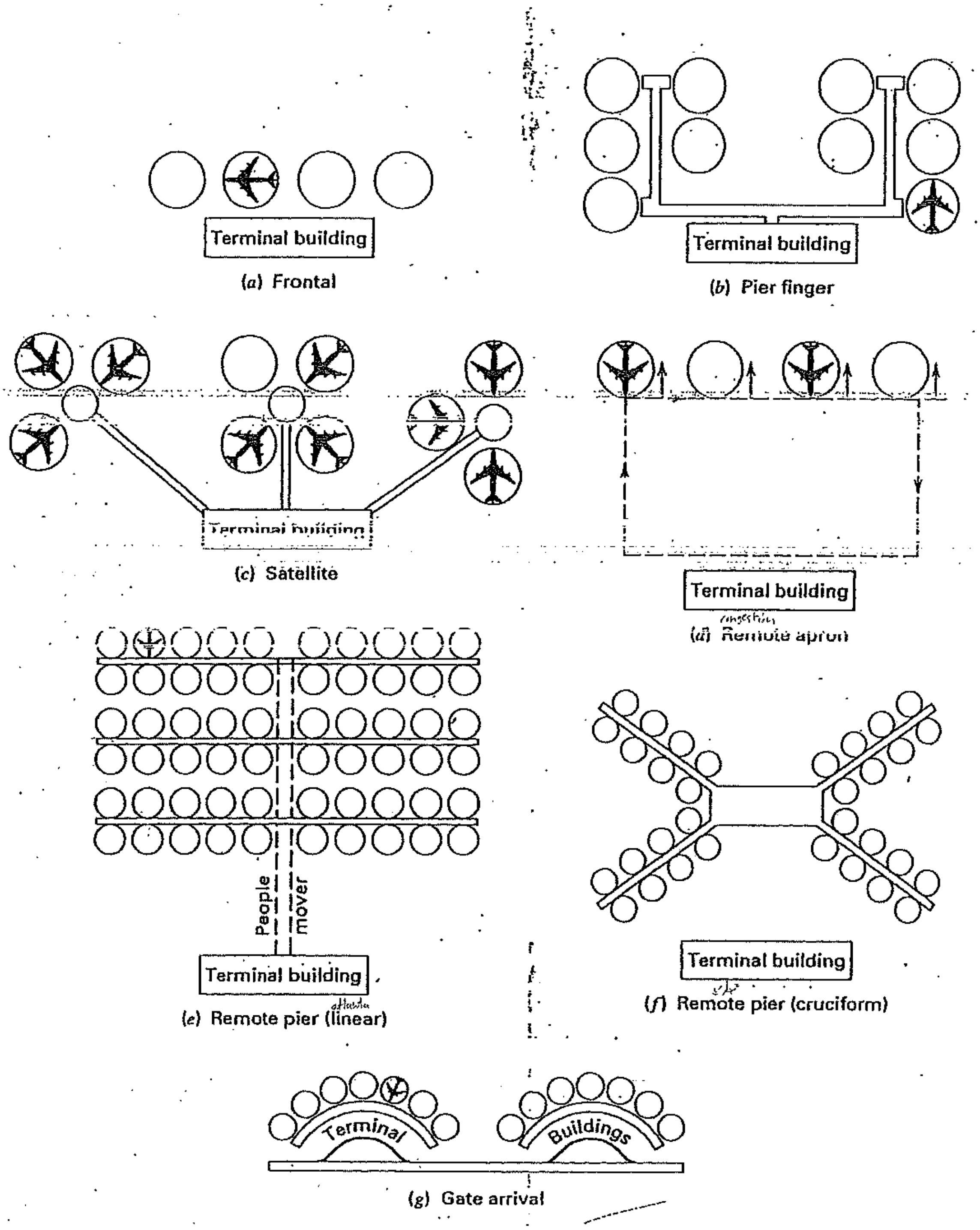


Figure 17-2 Terminal layout schemes.

craft served is small and where there are few or no flights by commercial air carriers. The aircraft are self-maneuvering in this arrangement.

A second layout scheme, shown in Fig. 17-2*b*, involves the use of concourse or pier *fingers* that extend from the terminal building to the aircraft parking areas. These fingers may simply be fenced walkways but commonly are enclosed structures that are temperature controlled throughout. Enclosed concourses protect passengers from the elements, aircraft noise, and propeller and jet blast.

In the mid-1950s, airlines began to process the passengers at the gate by accepting their tickets at a holding area within the terminal concourse structure. Later, when it became the practice to increase these areas in size and to provide comfortable furnishings, they came to be known as departure lounges.

Commenting on the departure lounge loading concept, it was once stated that "aside from the increase in the number of aircraft gates required for the increased schedules, no other single development had such an impact on terminal design (and, incidentally, cost) as this did" [3].

A logical extension of the departure lounge in the pier finger concept was the development of *satellite enplaning structures* (see Fig. 17-2*c*). The satellite structures are self-contained areas with departure lounges, toilets, concessions, and limited food and beverage areas. These structures typically serve 5 to 10 loading gates and are connected to the terminal buildings in a number of ways. In some cases the satellites were connected by a concourse with departure lounges along their length (e.g., Tullamarine, Melbourne), by a connecting tunnel beneath the apron (e.g., Los Angeles), or by a connector above the apron (e.g., Tampa).

As airports grow larger, the size of terminals with pier fingers and satellites become inconveniently large from the passengers' viewpoint, and the designs also require long taxiing distances for aircraft. The layouts shown in Figs. 17-2*d* and 17-2*e* are attempts to overcome this difficulty. The transporter or remote apron airport separates the servicing apron of the aircraft from the passenger terminal. Passengers are brought to and from the aircraft by transporters that can range from inexpensive buses to elaborate mobile lounges that can move up or down to match aircraft or terminal floor heights. This form of design was first used at Dulles Airport, Washington, D.C., and subsequently has been adapted elsewhere (e.g., Mirabel, Montreal, and Jeddah). Because of delay problems with disembarking and embarking passengers by mobile lounges, transporter airports are not popular with the airlines, which see this type of design as a cause of extended ground turnaround times. As airports grow even larger, this concept is modified to the remote pier arrangement exemplified by the Atlanta midfield terminal, where a high-capacity people-mover system transports passengers from the main terminal building to a series of linear gate buildings to which the aircraft taxi for service and turnaround (see Fig. 17-2*e*). A similar design has been constructed at Pittsburgh, where the remote pier is of a huge cruciform layout, shown in Figure 17-2*f*.

The automobile is the predominant mode of access for many large airports throughout the world, especially in the developed countries of Western Europe and North America. Where passenger access is predominantly by the automobile, designers have developed the *gate arrival* concept, which minimizes the walking distance between the point at which the air travelers park their cars or from where they are dropped off from either a car or taxi at the aircraft gate. This concept, shown in Fig. 17-2*g*, employs a very long terminal building that is a concourse or finger with aircraft loading positions on one side and the terminal roadway, curb, and public entrances on the other. Because of its length, special ground transportation facilities must be provided for passengers moving from one point in the terminal to another or between separate terminals in the airport terminal complex. In Kansas City shuttle buses are used, but in

Figure 17-3 Dallas-Fort Worth International Airport. (Courtesy of Dallas-Fort Worth International Airport.)

the larger Dallas–Fort Worth terminal a complicated automatic light guideway transit system is in operation.

With the gate arrival system, convenient automobile parking is provided, and abundant curb lengths are available for discharging and picking up passengers. There are, however, a number of disadvantages. There is some loss of cross-utilization of space and facilities. More rest rooms, concessions, and so on, are required than for centralized terminal designs. Where there are a large number of interlines at the airport, the passenger's difficulty in transferring makes this design unacceptably difficult. Figure 17-3 shows Dallas–Fort Worth International Airport, a gate arrival design. Gate arrival systems were more popular in times when airport terminal security provided no problems. They are security manpower intensive.

Many large airports have adopted the *unit terminal* approach, in which each airline has its own terminal building. An early example of this is the John F. Kennedy Airport in New York City, where each terminal is uniquely designed. Later examples of the unit terminal design where each unit was to conform to a similar design were Houston International Airport, Lester Pearson International Airport Toronto, and Paris Charles DeGaulle (see Fig. 17-4). In each case, the original circular unit concept was abandoned for a linear second terminal that was found to give more flexibility.

Richard de Neufville has correctly pointed out that airport designers frequently strive for overall designs that are aesthetically pleasing and symmetrical when viewed from the air [4]. In fact, designs should be functional, and the most successful operations may be achieved by assymetrical and seemingly piecemeal designs that are tailored to the needs of the individual air traffic demands of different airline operations [5, 6]. Seattle is an example of an airport with a terminal that combines the open apron, satellite, and pier finger concepts.

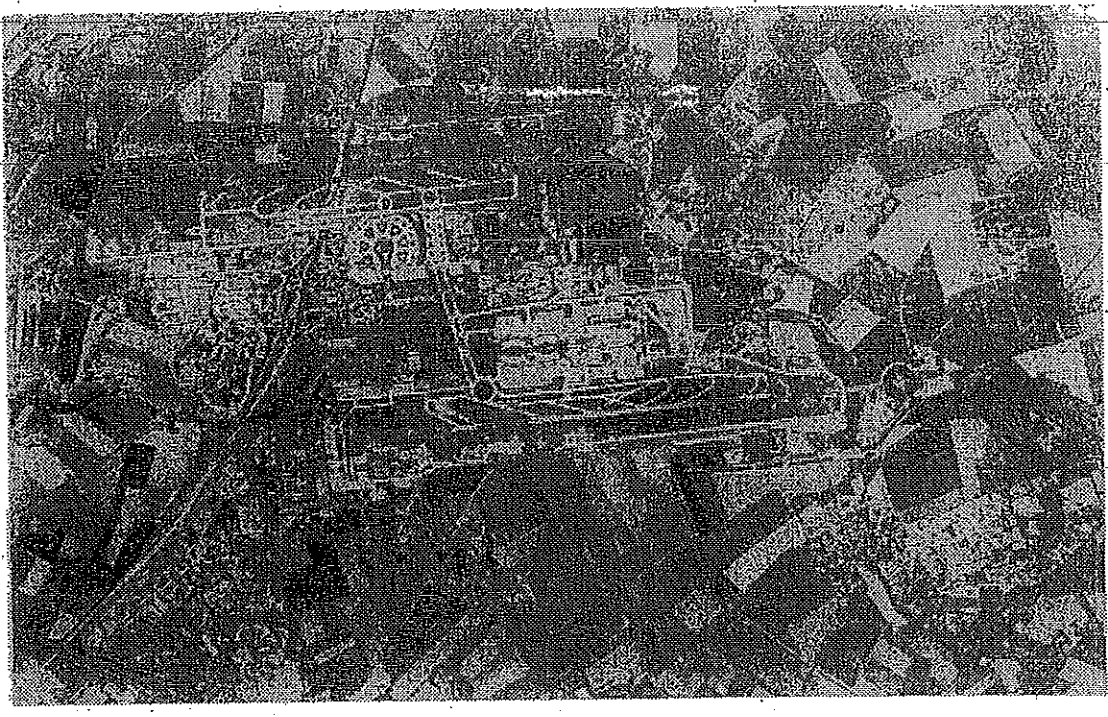


Figure 17-4 Aerial photograph of Charles de Gaulle International Airport, Paris. (Courtesy of Charles de Gaulle International Airport.)

less walking

17-5. INTRATERMINAL AND INTERTERMINAL TRANSPORTATION

Despite efforts to minimize walking distance by thoughtful air terminal layout and design, distance remains a problem at existing airports. As early as 1976, a study [7] of nine major airports[1] revealed that for originating passengers the average maximum walking distance to the nearest gate was 565 ft, while the minimum walking distance to the farthest gate was 1342 ft. Passengers who transferred from one airline to another were confronted with an average walking distance of 4091 ft. In the subsequent 20 years, airport terminals have grown very much larger and the problem has become more severe. The International Air Transport Association (IATA) recommends that the major walking distances such as from the parking lot to check-in and check-in to the boarding gate be limited to 300 m (1000 ft).

To overcome the problem of excessive walking distances, two basic approaches have been used: moving sidewalks and vehicle systems.

Moving sidewalks, which are usually limited to distances of about 300 m (1000 ft) due to their slow speed, have been used at such major airports as San Francisco, London Heathrow, and Paris Charles-de-Gaulle. They are used principally to connect parking lots to the terminal and to connect central processing areas to the departure lounges. There are two types of moving sidewalks: belt conveyors and horizontal escalators using linked pallets. To prevent mishap, handrails must be provided that must move at the same speed as the sidewalk.

A wide variety of vehicular systems have been developed or are proposed for transportation within the terminal area. Conventional shuttle buses are used extensively on

[1]Chicago (O'Hare), New York (John F. Kennedy), Los Angeles International, Atlanta, San Francisco International, Dallas, Miami International, Philadelphia International, and Detroit Metropolitan Airport.

Figure 17-5 Automated people-mover system at the Dallas-Fort Worth International Airport. (Courtesy Dallas-Fort Worth International Airport .

both the landside and airside of many terminals. Specially designed buses are used at a number of European and Far Eastern airports. Automatic transit vehicles are used to convey passengers to satellite terminals at Pittsburgh, Miami, Atlanta, and Tampa, for example. Movement between terminals is provided by an automatic computer-controlled light guideway system at Dallas–Fort Worth (see Fig. 17-5). Automatic systems normally are chosen to eliminate labor costs and to increase reliability. They are possible only when grade-separated designs are possible.

Perhaps the most dramatic example of specialized vehicles for transportation in the terminal area is the mobile lounge that was originally developed for Dulles International Airport, Washington, D.C., and subsequently has been used in such airports as Baltimore, Kennedy, Mirabel, and Madrid. Mobile lounges are essentially holding areas on wheels that transport passengers from terminal buildings to the aircraft. These vehicles are about 15 m (50 ft) long, 4.5 m (15 ft) wide, with a seating capacity of 60 persons. They can be driven from either end. Most versions of mobile lounges have bodies that can be raised or lowered to match aircraft floors or passenger loading ramps. Because of their complexity, these vehicles are extremely expensive, and an added disadvantage is their operational slowness, which has already been stated. However, studies have shown that where facilities are used sporadically in peak conditions it may be more economic to use mobile lounges rather than permanent buildings [4].

17-6. AUTOMOTIVE PARKING AND CIRCULATION NEEDS

Increasing airline activity and the growing popularity of the automobile have combined to create unprecedented demand for parking and circulation facilities at most airports. In small- and medium-sized cities, more than half of the air passengers travel to and from the airport by passenger car. Although this percentage may be only 20 to 25 percent for major airports, the total volume of passenger car traffic to these airports taxes the ingenuity of the airport designer in his efforts to provide adequate and convenient parking and circulation needs.

Ideally, the airline passenger, who is usually carrying baggage, should be provided a parking space within 90 to 120 m (300 to 400 ft) of the terminal building. A maximum walking distance from parking lot to the terminal building of 300 m (1000 ft) is recommended by IATA [5].

The classic parking plan has been a ground-level parking lot adjacent to the airport passenger terminal. At large airports, designers have found it impossible to provide sufficient parking on a single level, and the trend is toward the provision of multilevel parking structures. The design of these structures is similar to the design of a downtown parking garage (described in Chapter 15), the principal differences being the need to provide shorter walking distances and larger stalls for passengers carrying baggage. Parking garages at airports also tend to be larger than downtown garages, and several large airports are now designing ultimate garage capacities of 5000 to 12,000 spaces.

Most new U.S. airport parking facilities provide about 1000 parking spaces per million originating and destined annual passengers. Typical parking stalls are 2.6 to 2.7 m (8.5 to 9.0 ft) in width and 5.5 m (18 ft) long. A popular design features angle parking (60 degrees) on each side of a central one-way aisle 6.7 m (22 ft) in width. Thus, about 25.5 m^2 (275 ft^2) of net parking area, including the aisle, is required. When space needs of baggage dropoff and pickup areas, sidewalks, elevators, and stairs are included, approximately 32 m^2 (340 ft^2) per stall may be required.

Airport parking demand may be divided into several categories:

1. Passengers.
2. Visitors including those bringing passengers and well-wishers
3. Employees
4. Business callers
5. Rental cars, taxis, limousines, and so on

Since the parking characteristics of parkers in these various categories differ in time of occurrence and duration, separate parking analyses should be made for each category.

Preferably, parking spaces for short-time parkers should be located nearest the terminal, and airports charge higher fees for close-in parking spaces. At large airports, it is now common to have short-term parking close to the terminal and remote parking served by frequent shuttle buses at some distance from the terminal, in some cases even off the airport site. It is the practice for most large airports to separate parking spaces for employees some distance from the terminal building. Shuttle buses are likely to be required to transport employees to their destinations.

If possible, the airport should have direct connections to a controlled-access highway system. Within the airport, vehicular circulation is generally counterclockwise and one way. This permits passengers to be loaded and discharged safely from the right side of the vehicle. At-grade intersections should be avoided in the circulation system, and traffic should be separated by destination at the earliest possible point. The use of overhead or tunnel crossings should be considered to prevent mixing of pedestrian and vehicular traffic.

Airports served by road modes also have a need for pickup and dropdown curb space. At U.S. airports it is usual to provide approximately 107 m (350 linear ft) of curb space per million noninterlining passengers [6].

17-7. TERMINAL APRON SPACE REQUIREMENTS

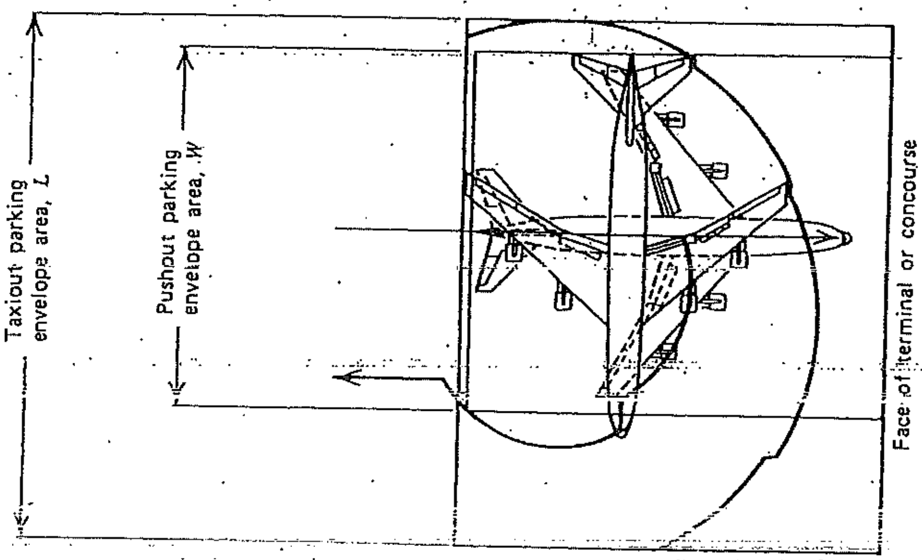
The term *apron* or *ramp* refers to an area for the parking or holding of aircraft. In terms of operational efficiency of the airport, the terminal apron, which is situated adjacent to the terminal building, is most important.

There are three primary factors that determine the space requirements for a passenger terminal apron:

1. Size of stand positions
2. Number of stand positions
3. Aircraft parking configuration

Size of Stand Positions. The size of stand positions is principally determined by the size and maneuverability of aircraft, but it is also influenced by desirable wing-tip clearances and the manner in which the aircraft is moved into the gate position and serviced. Table 17-1 indicates the recommended apron dimensions required for maneuvering aircraft of various sizes, based on aircraft dimensions of the mid-1990s [7].

Table 17-1 Diagram and Summary of Push-Out and Taxi-Out Dimensions



truck

self power

Group A/C	Push-Out[a] L	Push-Out[a] W	Area (yd²)	Taxi-Out[b] L	Taxi-Out[b] W	Area (yd²)
A. FH-227	103 ft 1 in.	115 ft 2 in.	1319	148 ft 10 in.	140 ft 2 in.	2318
YS-11B	106 ft 3 in.	124 ft 11 in.	1474	171 ft 0 in.	149 ft 11 in.	2850
BAC-111	123 ft 6 in.	113 ft 6 in.	1557	130 ft 0 in.	138 ft 6 in.	2001
DC9-10	134 ft 5 in.	109 ft 5 in.	1634	149 ft 2 in.	134 ft 5 in.	2228
B. DC9-21, 30	149 ft 4 in.	113 ft 4 in.	1880	149 ft 0 in.	138 ft 4 in.	2290
727 (all)	173 ft 2 in.	128 ft 0 in.	2463	194 ft 0 in.	153 ft 0 in.	3298
737 (all)	120 ft 0 in.	113 ft 0 in.	1507	145 ft 4 in.	138 ft 0 in.	2228
C. B-707 (all)	172 ft 11 in.	165 ft 9 in.	3188	258 ft 0 in.	190 ft 9 in.	5468
B-720	156 ft 9 in.	150 ft 10 in.	2627	228 ft 0 in.	175 ft 10 in.	4454
D. DC-8-43, 51	170 ft 9 in.	162 ft 5 in.	3081	211 ft 10 in.	187 ft 5 in.	4411
DC 8-61, 63	207 ft 5 in.	168 ft 5 in.	3882	252 ft 4 in.	193 ft 5 in.	5423
E. L-1011	188 ft 8 in.	175 ft 4 in.	3676	263 ft 6 in.	200 ft 4 in.	5865
DC10	192 ft 3 in.	185 ft 4 in.	3959	291 ft 0 in.	210 ft 4 in.	6801
F. B-747	241 ft 10 in.	215 ft 8 in.	5795	328 ft 0 in.	240 ft 8 in.	8771

[a] Including clearances of 20-ft wing-tip, nose to building; 30-ft group A and B, 20-ft group C and D, 10-ft group E and F.

[b] Including clearance of 20-ft to other A/C and GSE: 45 ft.

Note: Length and width are based on the largest dimension in the group of aircraft.

The amount of space required for maneuvering and serving the aircraft will vary depending upon airline operational procedures. The airport engineer should, therefore, consult with the various airlines on this matter during the early phase of the design process.

Number of Stand Positions. The number of stand positions required depends on (1) the peak volume of aircraft to be served and (2) how long each aircraft occupies a stand position. Stand occupancy time will depend on:

1. Type of aircraft
2. Number of deplaning and enplaning passengers
3. Amount of baggage
4. Magnitude and nature of other services required
5. Efficiency of apron personnel

From first principles, it can be deduced that the peak number of stand positions required at an airport is a function of the design peak-hour aircraft movements, the length of time that the individual aircraft spend at the stand, and some utilization factor that takes into account the impossibility of having all stands filled for 100 percent of the peak period and the lack of suitability of all aircraft to all stands.

One formula that has been proposed for computing the required number of stands is [8]:

$$n = \frac{vt}{u}$$

USA

where

v = design hour volume for departures *or* arrivals, aircraft/hr (the higher one)
t = weighted mean stand occupancy, hr
u = utilization factor, suggested to be 0.6 to 0.8 where stands are shared - according to efficiency of country

Another deterministic formula that has been calibrated on European traffic is [9]

$$n = mqt$$

where

m = design hour volume for arrivals *and* departures, aircraft/hr
q = proportion of arrivals to total movements
t = mean stand occupancy, hr

Computation of stand requirements for an individual airport is now carried out frequently by computer simulation models that take into account the specific design of the apron area, the mix of traffic, and the handling times likely to be achieved in practice. Recently work has been carried out to generalize the work of simulation models in this area [10].

A study [11] of nine airports indicated that there was wide variation in the time and magnitude of peak stand occupancy, but each airport had a definite pattern that remained relatively constant over the years. It was further reported that there was a wide range of productivity per stand for various airports. The following formula was proposed for forecasting the number of future stand positions required for a given airport:

$$\text{Future stands} = \left[(\text{present stands} - 2) \times \frac{\text{future passengers}}{\text{present passengers}}\right] + 2$$

This equation is applicable to any group of aircraft that has mutual use of stands, and separate calculations should be made for each such group. For example, if four groups of five-stands are considered separated for traffic and the future/present traffic ratio is 3, for each group,

$$\text{Future stands} = (5 - 2) \times 3 + 2 = 11$$

Thus, the total stands required for four groups will be 44. It is recommended that in actual practice approximately 15 percent be added to the number of stands computed by the above formula to allow for contingencies of operations such as early arrivals or delayed departures.

This approach can be used only when there is expected to be no major change in airline operating procedures in the future.

Aircraft Parking Configuration. Parking configuration refers to the orientation of aircraft in relation to the adjacent building when the aircraft is parked. There are a variety of parking configurations that may be used. The aircraft may be nosed-in, nosed-out, parked parallel, or at some angle to the building or concourse. The parallel parking system, in which the longitudinal axis of the aircraft is parallel to the adjacent building, has been used extensively and provides a good configuration for passenger flow. At major airports, because of demonstrated passenger preference, the trend is toward the use of a nose-in parking configuration with the aircraft being pushed away from the loading bridge or gangplank by a tractor after it is loaded. This parking configuration is especially suitable for use with circular satellite enplaning structures. In this case, the aircraft parks in the wedge. This permits flexibility in the provision of space for various sizes of aircraft since the amount of available space can be varied by moving aircraft out from the center of the satellite. This will imply, of course, air bridge reconfiguration.

17-8. THE TERMINAL BUILDING: TRANSPORT AIRPORTS

A well-designed terminal building is a vital element in the successful operation of an air carrier airport. It must provide for the smooth and efficient transfer of passengers and their baggage between surface transportation vehicles and aircraft.

The terminal building must provide ordered space and facilities for a variety of functions relating to air passenger service, air carrier operations, and operation and maintenance of the airport.

Required Air Passenger Service Functions. The terminal building usually houses a variety of air passenger service functions, including ticket sales, restroom services, waiting and resting, and baggage checking and claiming. In addition, terminal facilities will normally be required for security protection, provision of flight information, passenger boarding and deplaning, and the handling and processing of mail and light cargo.

Additional Facilities for Convenience of Passengers. Most moderate-size to large airports provide numerous and varied facilities for the convenience of passengers and to generate net airport income. Such facilities usually include a newsstand, telephones, restaurants and coffee shops, gift shops, insurance sales, and car rental agencies. Large airports typically also have a bank, a barber shop, medical services, and hotel and motel accommodations.

Air Carrier Operations. Consideration must also be given to space requirements in the terminal building for air carrier operations, including (1) a communications center, (2) ground crew and air crew ready rooms, and (3) an operations room for crews.

Airport Operations and Maintenance. Finally, space may be required in the terminal building for the functions relating to the operation and maintenance of the airport. Certain of these functions may be housed in separate buildings:

1. Air traffic control
2. Ground traffic control
3. Airport administration
4. FAA and other governmental administrative functions
5. Airport maintenance
6. Fire protection
7. Employee cafeterias
8. Utilities

In view of the rapidly expanding and changing nature of air travel activity, it is especially important that terminal buildings be planned and designed to allow easy expansion and change. Generally, a rectangular configuration rather than an odd-shaped building is preferred. The design should be such that it will not be necessary to relocate kitchen facilities, toilets, and other such costly installations should expansion be required. Nonbearing partitions should be used whenever possible to allow for reallocation of space to meet changing requirements. In short, the terminal building design should be expansible and flexible, and long-range plans should be made to change it and add to it as traffic and economic conditions dictate. Because of diseconomies of scale, terminal buildings should be built in a modular manner with modules designed to handle annual throughputs of between 5 and 10 million passengers. Single-terminal units begin to become very large with long walking distances when designed to handle in excess of 20 million passengers [12].

The terminal building design should provide for separation of service areas to prevent passenger and baggage congestion. Specifically, the lobby and waiting room activities should be separated from baggage-handling activities.

The design should provide for ease of circulation of enplaning and deplaning passengers. Enplaning passengers should be able to move directly and smoothly to the ticket counter, thence through the waiting room area to the aircraft loading gate. Deplaning passengers should be able to follow a direct route from the aircraft to the baggage claim area and thence to the passenger loading platform. These movements are illustrated schematically in Fig. 17-6 for an airport with decentralized security and gate control and check-in.

In the design of high-capacity terminal buildings, consideration should be given to the use of two- or three-level circulation systems. By providing two or more levels in the terminal building, a vertical separation of passenger and baggage flow can be realized. A multiple-level building also makes it easier to separate arriving and departing passengers and to provide dropoff and pickup curb space on the land side of the terminal.

Space Requirements for the Terminal Building. Before one can obtain a reliable estimate of space requirements for the various functions of the terminal building, it is necessary to estimate the typical peak-hour passengers.[2] In the case of general aviation airports, typical peak-hour passengers have been shown to depend on the number of hourly aircraft operations [1]. (See Fig. 17-7.) At the larger air carrier airports, the number of peak-hour passengers can be obtained from Fig. 17-8, which relates typical peak-hour passengers to total annual passengers.

[2]Typical peak-hour passengers are defined as the total of the highest number of passengers enplaning and deplaning during the busiest hour of a busy day of a typical week.

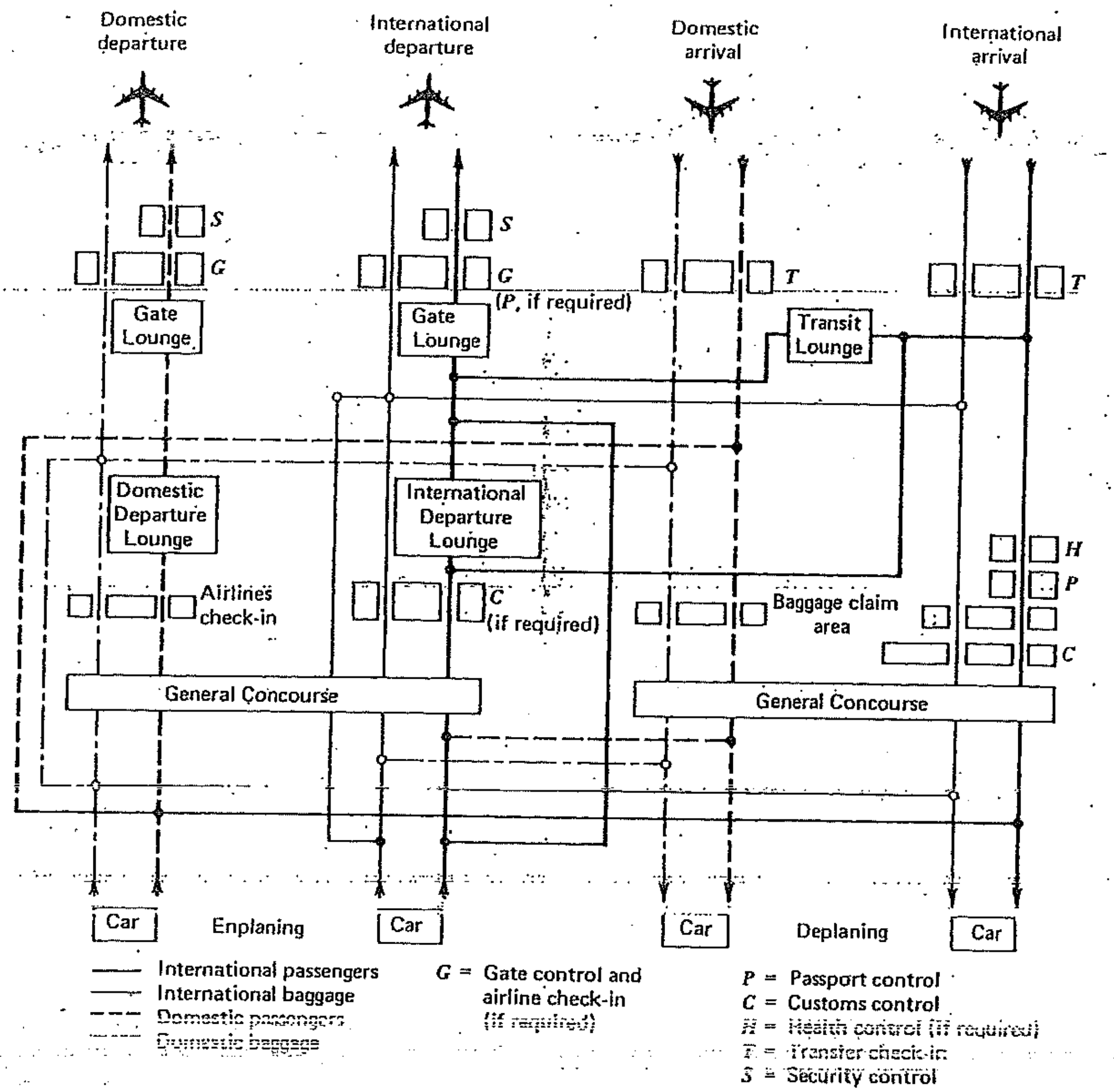


Figure 17-6 Passenger baggage flow system.

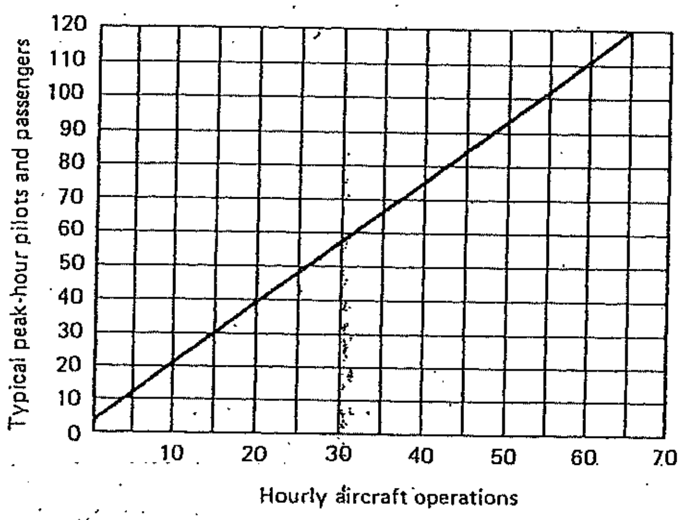


Figure 17-7 The relationship between typical peak-hour pilots and passengers and hourly aircraft operations at general aviation airports. (Source: Federal Aviation Administration.)

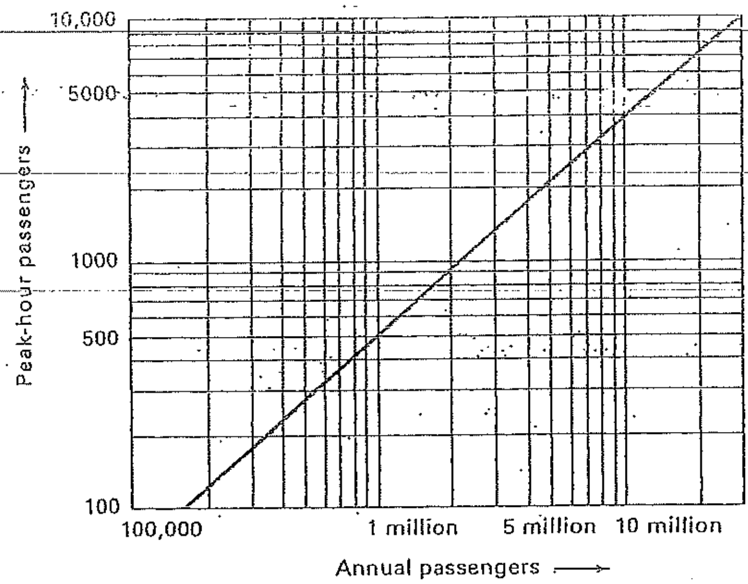


Figure 17-8 Relation between peak-hour and annual passenger flows.

More precise estimates of the peak-hour population of airports can be obtained by making actual counts of passengers, employees, visitors, and customers making use of existing terminal facilities and estimating future populations using FAA step-down passenger forecasts of future passenger demand for individual airport facilities.

The FAA has published design recommendations to aid those planning terminal buildings [7]. Using this publication, space requirements for the various activities in the terminal buildings may be estimated from graphs relating space needs to typical peak-hour passenger throughput. While the graphs are useful for planning purposes, more detailed study may be required for design purposes to allow for local variations from average conditions [9].

Table 17-2 gives passenger space requirements according to the IATA-based design standards, which have been adopted and in some cases slightly modified by a number of airport authorities [13].

17-9. THE GENERAL AVIATION TERMINAL AREA [1]

The requirements for general aviation terminal areas are relatively simple. Initially where there is little aviation activity, a small maintenance hangar حظيرة طائرات with an attached office will cater for the needs of the facility. A separate administrative terminal building should be built only when there are:

a. A minimum traffic volume of 10 operations, excluding touch and go, during the peak hours of a typically busy day
b. One or more fixed-based operators on the airport
c. Airplane fuel available on the airport
d. A hangar with repair facilities in operation
e. A full-time manager on duty during the normal day
f. A need for a public waiting area and restrooms along with a telephone

Before engaging in the design of a general aviation terminal, a survey should be carried تنفيز out to determine peak demand. For such terminals, this is the greatest number of pilots and passengers enplaning and deplaning during the busiest hour of the busy day of a typical week, *not* the absolute peak occurring on an abnormally busy day.

The terminal and administration building should be functional and designed to permit

Table 17-2 Passenger Terminal Space Standards[a]

Facility	Space Standard	Time Standard
Check-in baggage drop	0.8 m^2/passenger with baggage 0.6 m^2 for visitors	95 percent of passengers <3 min; at peak times 80 percent <5 min
Departure concourse	None	None
Departure passport control	0.6 m^2/passenger without hold baggage 0.8 m^2/passenger with hold baggage	95 percent of passengers <1 min
Central security	—	95 percent of passengers <3 min; for high-security flights, 80 percent <8 min
Departure lounge	1.0–1.5 m^2/seated passenger 1.2 m^2/standing passenger with trolley 1.0 m^2/standing passenger Seating for 50 percent of throughput	
Gate lounge	0.6 m^2 for queuing passenger without hold baggage 0.8 m^2 for queuing passenger with hold baggage 1.0 m^2/passenger within gate lounge	80 percent should queue less than 5 min for gate check-in
Immigration	0.6 m^2/passenger	95 percent of all passengers <12 min; 80 percent of nationals <5 min
Baggage reclaim	0.8 m^2/domestic and short-haul international passenger 1.6 m^2 for long-haul passenger	Maximum of 25 min from first passenger in hall to last baggage from unit 90 percent of passengers wait <20 min for baggage
Customs	2.0 m^2/passenger interviewed	None
Arrivals concourse	0.6 m^2/standing meeter; 1.0 m^2/seated meeter 0.8 m^2/short-haul passenger, 1.6 m^2/long-haul passenger	None

[a]Additional standards:

Forecourts	95 percent chance of finding space.
Piers	Walking distances: <250 m unaided; <650 m with walkway (of which 200 m unaided) Rapid transit for point-to-point journeys over 500 m
Pier service	Loading bridges for at least 75 percent of passengers

Source: Reference 13.

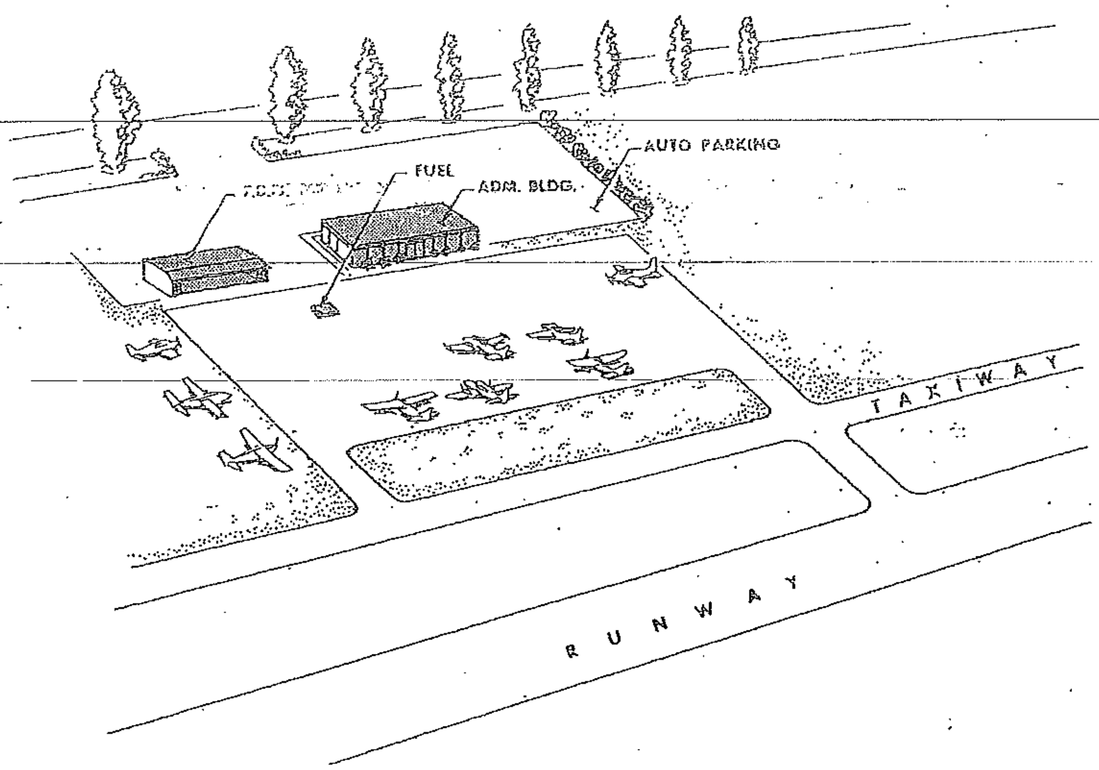
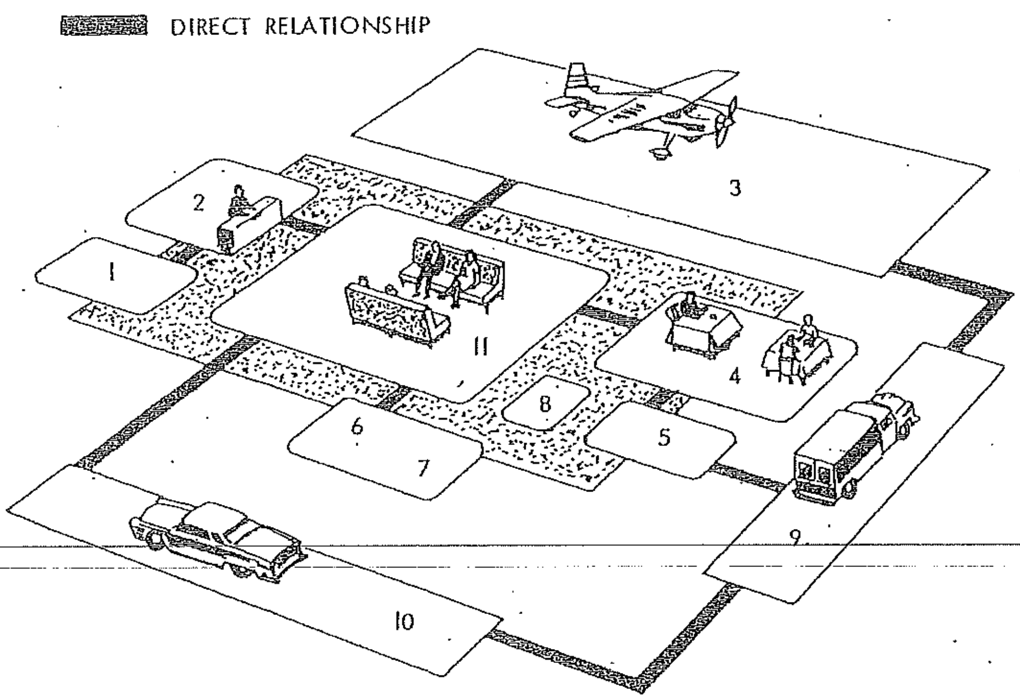


Figure 17-9 Apron terminal area at a general aviation airport. (Courtesy Federal Aviation Administration.)



1. STORAGE
2. OPERATION MANAGEMENT
3. AIRPLANE LOADING APRON
4. DINING AREA
5. KITCHEN
6. REST ROOMS
7. JANITOR CLOSET
8. UTILITIES
9. SERVICE AND APRON ACCESS DRIVE
10. ADMINISTRATION BUILDING DRIVE
11. WAITING AREA

Figure 17-10 A general aviation terminal/administration building. (Courtesy Federal Aviation Administration.)

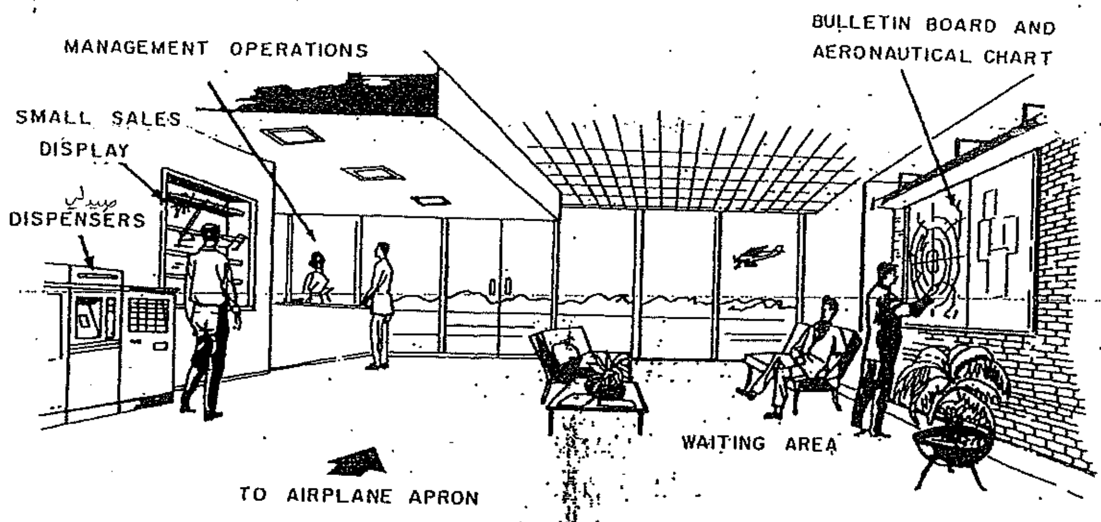


Figure 17-11 Waiting area in general aviation terminal. (Courtesy Federal Aviation Administration.)

easy expansion. Figure 17-9 shows the simple juxtaposition of the runway, taxiway, apron, and terminal building at a general aviation facility, and Fig. 17-10 indicates how building and access routes can be positioned for a simple functional layout that minimizes walking distance.

Minimum terminal size is likely to be in the region of 232 m^2 (2500 ft^2), with the airport manager's office taking up about 17 m^2 (180 ft^2) and having a good view of the airfield operating area.

The waiting room, which should have a minimum size of 93 m^2 (1000 ft^2) should also overlook the airfield and should provide comfortable seating arrangements. All buildings should have some concessions, which at small facilities may be coin operated. There should also be a bulletin board for weather reports and notices to airmen as well as a space for mounting aeronautical charts. At larger terminals, eating facilities in the form of a dining room should be provided, in addition to public restrooms and a telephone. On the landside of the terminal, there should be adequate parking and road access provision. Figure 17-11 shows the inside of the waiting area of a medium-sized general aviation terminal.

17-10. SUMMARY

An airport terminal area is a complex and delicately balanced microcosm. Inevitable changes in passenger loads, actions by the airlines and airport management, and technological developments tend to upset this balance, sometimes resulting in undesirable and near-intolerable passenger and aircraft congestion and delays. Airport passenger loads typically fluctuate in the short run and increase in the long run. Airline companies react to heavier traffic by instituting innovations in passenger and baggage-handling procedures and facilities. The introduction of larger and faster aircraft creates a need for more and larger waiting rooms and baggage pickup areas and may overload auto parking and circulation facilities. Airport planners and designers accept an awesome challenge in attempting to provide terminal facilities that resist obsolescence in the face of constant change.

PROBLEMS

1. Compute the required number of aircraft gates from the following information:

Number of arrivals in the peak hour	25
Average stand occupancy time	45 min
Utilization factor	0.8
Proportion of arrivals to total movements	0.6

2. The gate arrival system of airport terminal design has been promoted extensively as a good solution for the airports of the United States. Discuss how well this solution would work for airports with a large proportion of international and transfer passengers.
3. Sketch in schematic form the layout of a combined passenger terminal and administration building for a general aviation airport with 50 hourly airplane operations. Consult reference 1 when preparing this answer.

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[illegible] apron space 561

Chapter 18

Airport Design Standards and Procedures

18-1. INTRODUCTION

In many respects, this chapter is the most important of the three chapters that deal with air transportation. It presents specific design standards and procedures that are required for the preparation of plans and specifications for an airport. Topics covered in this chapter will include runway lengths, geometric design of the runway system, earthwork, drainage, paving, and lighting and marking.

It should be remembered that the design standards given in this chapter are recommended standards rather than absolute requirements. Developed by the FAA for all parts of the nation, the standards are based on broad considerations. Local conditions and requirements must justify deviation from a particular standard in order to secure an advantage relating to another design feature. In such a case, designers should be prepared to justify their decisions to deviate from an accepted engineering design standard. In any event, designers would be well advised to check with the nearest office of the FAA.

18-2. RUNWAY LENGTH

One of the most important design features for an airport is runway length. Its importance stems from its dominant influence on air safety and size and cost of the airport.

Design runway length is influenced most by the performance requirements of the aircraft using the airport, especially when operated with its maximum landing and take-off loads. Variations in required runway length are caused by:

air density

1. Elevation of the airport
2. Average maximum air temperature at the airport . T↑ .. density ↓
3. Runway gradient

Most IMP

~~4. Design aircraft~~ (4) Design of aircraft

The FAA provides a family of curves for small airports for aircraft with a maximum certificated take-off weight of 5670 kg (12,500 lb) or less with approach speeds of 50 knots or more [1]. Figures 18-1 and 18-2 give the FAA design curves for runways serving small airplanes with less than 10 passenger seats and 10 or more passenger seats, respectively.

less than 12,500 lb

$T_F = (T_c * \frac{9}{5}) + 32$

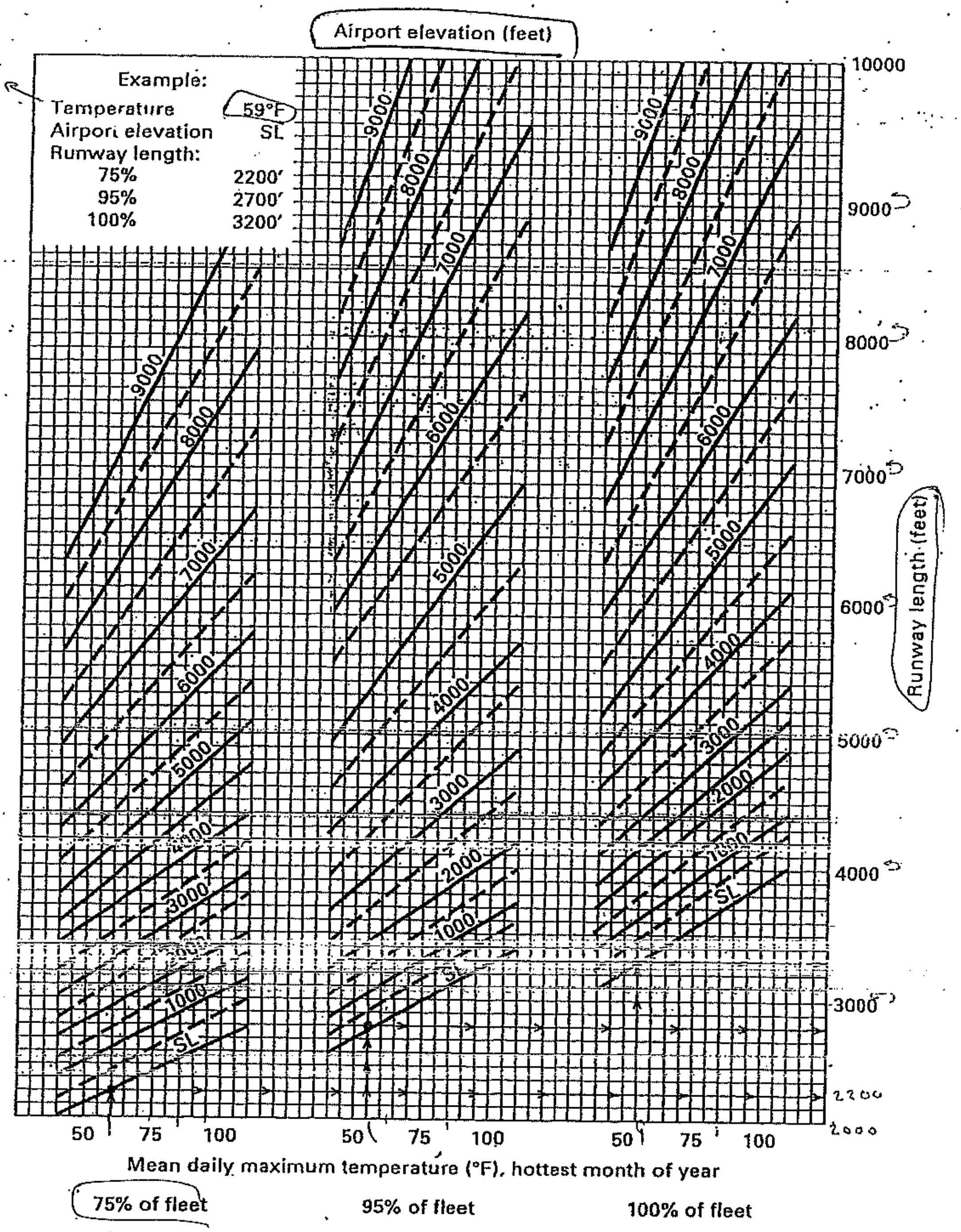


Figure 18-1 Runway lengths required to serve small airplanes having less than 10 passenger seats. (Source: *Runway Length Requirements for Airport Design*, FAA Advisory Circular 150/5325-4A, Change 1, March 11, 1991.)

To use the curves in Figs. 18-1 and 18-2, one should enter the appropriate family of curves on the abscissa axis at the normal maximum temperatures.[1] From this point, a line is extended vertically until it intersects the slanted line corresponding to the airport elevation, interpolating if necessary. The point of intersection is extended horizontally to the right ordinate where the required runway length can be read.

[1]The normal maximum temperature is defined as the arithmetical average of the daily highest temperature during the hottest month. This information may be obtained from the National Oceanic and Atmospheric Administration National Climatic Center, Asheville, NC, Technical Paper No. 81.

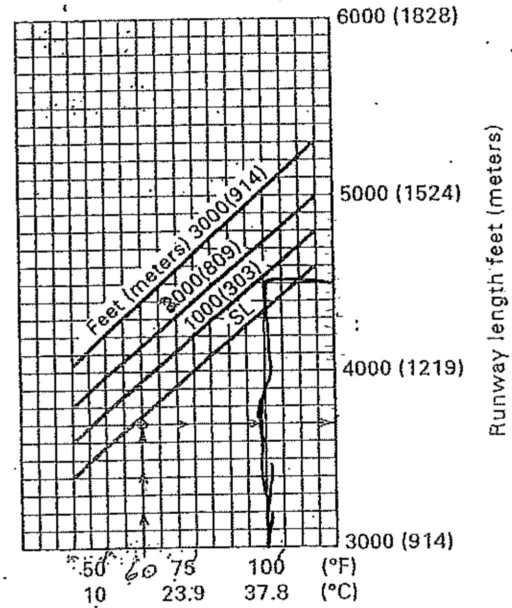
Representative airplanes	Runway length curves
Beech B80 Queen Air Beech E90 King Air Beech B99 Airliner Beech A100 King Air Britten-Norman Mark III-I Trilander	Example: Temperature 59°F (15°C) Airport elevation Sea level Gen. utility 3700' (1128 m) Note: For airport elevations above 3000 feet (914m) use the 100% curve, figure 8-1
Mitsubishi MU-2L Swearigen Merlin III-A Swearigen Merlin IV-A Swearigen Metro II No extrapolation here, if > 3000 we go to fig. 18-1 & use 100%. (we can extrapolate below SL only)	Mean daily maximum temperature of the hottest month of the year
[illegible]	

Figure 18-2 Runway lengths required to serve small airplanes having 10 passenger seats or more. (Source: *Runway Length Requirements for Airport Design,* FAA Advisory Circular 150/5325-4A, Change 1, March 11, 1991.)

figures 18-1 & 18-2 – no correction required

EXAMPLE 18-1 *Use of Design Curves for Runways at Small Airports, I*

It is necessary to calculate the runway length requirement for a small airport at sea level where mean daily maximum temperature is 29.4°C (85°F). The aircraft using this runway will all have less than 10 passenger seats. Compute the runway lengths that will satisfy 75 percent, 95 percent, and 100 percent of the fleet.

Figure 18-1 is chosen because of the size of the aircraft seating. The figure is entered at the abscissa axis at 85°F and vertical lines are drawn until they reach the slanted-line curves. The intercept of these points on the vertical axis of the graphs gives the appropriate runway lengths. These are:

75 percent of fleet	2420 ft (748 m)
95 percent of fleet	3000 ft (929 m)
100 percent of fleet	3580 ft (1109 m)

All graphs may be interpolated.

EXAMPLE 18-2 *Use of Design Curves for Runways at Small Airports, II*

What length of runway is required for a small airport that is 305 m (1000 ft) above sea level and has a normal maximum temperature of 80°F (26°C)? The airport is to accommodate aircraft with a maximum certificated weight of less than 5670 kg (12,500 lb) and more than 10 passenger seats.

Select Figure 18-2, because of the number of seats, entering at an abscissa value of 80°F (26°C). Project a line vertically to intersect the 1000-ft curve. A required runway length of 1311 m (4300 ft) is indicated by extending the point of intersection horizontally. Where necessary the graph may be interpolated. The value obtained is the answer. No correction is required for runway gradient or other factors.

It is normal practice to design runway lengths at airports handling large aircraft around particular airplanes. The likely type of aircraft to be accommodated and a critical aircraft is chosen, around which the design is formulated. The FAA publication *Runway Length Requirements for Airport Design* [1] provides design tables for landing and take-off requirement for airplanes in common use in civil aviation. These tables are based on actual flight test and operational data.

Examples of FAA runway length tables are shown in Tables 18-1 and 18-2, which are based on the performance of a Boeing 757-232 series. These and similar performance curves and tables are based on an effective runway gradient of zero percent. Effective runway gradient is defined as the maximum difference in runway centerline elevations divided by the runway length. The FAA specifies that the runway lengths for take-off should be increased by 10 ft per foot of difference in centerline elevation between the high and low points of the runway centerline elevations.

EXAMPLE 18-3

Compute the runway length requirements for a Boeing 757-232 series aircraft landing and taking off at an airport at an elevation of 500 m (1640 ft) at a normal maximum temperature of 30°C (86°F). Assume 25° flaps for landing and 5° flaps for take-off. Maximum operational take-off weight is 102,000 kg (224,686 lb) and maximum operational landing weight is 84,000 kg (185,035 lb). The airplane has two Pratt & Whitney 2037 engines.

Landing runway requirement Referring to the FAA manual [1], the landing table shown in Table 18-1 would be used, reflecting the choice of 25° flaps and the PW engine 2037. Using the upper part of the table, entering with an airport elevation of 500 m and a temperature of 30°C, the maximum permissible landing weight is given as 89,800 kg (197,811 lb). The operational limit is below this and is therefore permissible.

Entering the lower part of the table with 185,035 lb and an elevation of 1640 ft, interpolation between 5.72 and 5.86 in the table gives 5.57 or 5570 ft (1389 m).

Take-off runway requirement For the take-off runway requirement, with 5° flaps and two PW 2037 engines the appropriate FAA table is shown in Table 18-2. Entering the top of the table with maximum temperature 30°C and elevation 500 m, the maximum permissible take-off weight is 108,900 kg (239,885 lb). This is greater than the maximum operational weight to be used and the operational weight is therefore permitted.

Entering the middle table with 30°C and 500 m elevation the reference factor R is found to be 59.7. This factor is then used to enter the lowest part of the table in combination with the operational weight of 102,000 kg. This gives 2581 m (8468 ft) by interpolation.

Table 18-1 Runway Length Table: Aircraft Performance, Landing (Boeing 757-232 Series) PW 2037 Engine, 25° Flaps

Temperature (°C)	By Airport Elevation in Meters: 0 m	500 m	1000 m	1500 m	2000 m	2500 m
	Maximum Allowable Landing Weight (1000 kg)					
10	89.8	89.8	89.8	89.8	89.8	89.8
12	89.8	89.8	89.8	89.8	89.8	89.8
14	89.8	89.8	89.8	89.8	89.8	89.8
16	89.8	89.8	89.8	89.8	89.8	89.8
18	89.8	89.8	89.8	89.8	89.8	89.8
20	89.8	89.8	89.8	89.8	89.8	89.8
22	89.8	89.8	89.8	89.8	89.8	89.8
24	89.8	89.8	89.8	89.8	89.8	89.8
26	89.8	89.8	89.8	89.8	89.8	89.8
28	89.8	89.8	89.8	89.8	89.8	89.8
30	89.8	89.8	89.8	89.8	89.8	89.8
32	89.8	89.8	89.8	89.8	89.8	89.8
34	89.8	89.8	89.8	89.8	89.8	89.8
36	89.8	89.8	89.8	89.8	89.8	89.0
38	89.8	89.8	89.8	89.8	89.8	86.9
40	89.8	89.8	89.8	89.8	89.8	84.9
42	89.8	89.8	89.8	89.8	87.5	82.9
44	89.8	89.8	89.8	89.8	85.2	81.0

Weight (1000 lb)	By Airport Elevation in Feet: 0 ft	1000 ft	2000 ft	3000 ft	4000 ft	5000 ft	6000 ft	7000 ft	8000 ft
	Runway Length (1000 ft)								
125	3.65	3.72	3.79	3.85	3.92	3.98	4.05	4.11	4.18
130	3.89	3.96	4.04	4.12	4.21	4.30	4.39	4.49	4.60
135	4.08	4.17	4.26	4.35	4.45	4.56	4.67	4.80	4.94
140	4.24	4.33	4.43	4.54	4.65	4.77	4.90	5.04	5.20
145	4.37	4.48	4.58	4.69	4.81	4.94	5.08	5.24	5.41
150	4.49	4.60	4.71	4.83	4.95	5.09	5.23	5.39	5.56
155	4.60	4.71	4.82	4.95	5.08	5.22	5.36	5.52	5.69
160	4.70	4.81	4.93	5.06	5.19	5.33	5.48	5.64	5.81
165	4.80	4.92	5.04	5.17	5.30	5.45	5.60	5.76	5.92
170	4.92	5.03	5.16	5.29	5.43	5.57	5.72	5.88	6.04
175	5.04	5.16	5.29	5.42	5.57	5.71	5.87	6.03	6.20
180	5.19	5.32	5.44	5.58	5.73	5.88	6.04	6.22	6.39
185	5.37	5.50	5.63	5.77	5.92	6.09	6.26	6.45	6.65
190	5.59	5.72	5.86	6.00	6.16	6.33	6.53	6.74	6.97
195	5.85	5.98	6.13	6.28	6.45	6.64	6.85	7.10	7.38
200	6.15	6.30	6.45	6.61	6.79	7.00	7.25	7.54	7.89

Interpolation:

Table 18-2 Runway Length Table: Aircraft Performance, Takeoff (Boeing 757-232 Series) PW 2037 Engine, 5° Flaps

Temperature (°C)	By Airport Elevation in Meters					
	0 m	500 m	1000 m	1500 m	2000 m	2500 m
	Maximum Allowable Takeoff Weight (1000 kg)					
10	108.9	108.9	108.9	108.9	108.9	106.4
12	108.9	108.9	108.9	108.9	108.9	104.9
14	108.9	108.9	108.9	108.9	108.8	103.4
16	108.9	108.9	108.9	108.9	107.7	101.9
18	108.9	108.9	108.9	108.9	106.4	100.3
20	108.9	108.9	108.9	108.9	105.0	98.6
22	108.9	108.9	108.9	108.9	103.4	97.0
24	108.9	108.9	108.9	107.9	101.7	95.3
26	108.9	108.9	108.9	106.0	100.0	93.5
28	108.9	108.9	108.9	104.1	98.1	91.7
30	108.9	108.9	107.5	102.1	96.1	89.9
32	108.9	108.9	105.5	100.0	94.1	88.0
34	108.9	108.4	103.4	97.8	91.9	86.1
36	108.9	106.5	101.3	95.6	89.7	84.1
38	108.9	104.5	99.0	93.2	87.4	82.1
40	104.5	106.5	96.7	90.8	85.1	80.0
42	105.3	99.9	94.1	88.3	82.7	77.8
44	103.0	97.3	91.4	85.7	80.3	75.6

Temperature (°C)	By Airport Elevation in Meters					
	0 m	500 m	1000 m	1500 m	2000 m	2500 m
	Reference Factor *R*					
10	52.1	54.8	58.2	62.7	68.5	76.1
12	52.2	54.7	58.2	62.9	69.1	77.0
14	52.3	54.8	58.3	63.2	69.8	78.2
16	52.4	54.9	58.6	63.7	70.6	79.6
18	52.6	55.2	59.0	64.4	71.7	81.2
20	52.8	55.6	59.6	65.3	72.9	82.9
22	53.1	56.2	60.4	66.3	74.4	84.9
24	53.5	56.8	61.4	67.6	76.0	87.2
26	53.9	57.6	62.5	69.1	78.0	89.6
28	54.5	58.6	63.8	70.8	80.1	92.4
30	55.1	59.7	65.4	72.8	82.6	95.3
32	56.0	61.0	67.1	75.0	85.3	98.5
34	56.9	62.4	69.0	77.5	88.3	102.0
36	58.0	64.0	71.2	80.2	91.6	105.8
38	59.3	65.7	73.6	83.3	95.2	109.9
40	60.7	67.7	76.2	86.6	99.1	114.2
42	62.4	69.8	79.0	90.2	103.4	118.8
44	64.2	72.1	82.1	94.1	108.1	123.8

Weight (1000 kg)	Runway Length in Meters (Reference Factor *R*)								
	60	70	80	90	100	110	120	130	140
60	941	1087	1245	1401	1540	1648	1711	1715	1646
65	1093	1259	1418	1569	1711	1844	1966	2076	2173

1,5   0,9~

Table 18-2 Runway Length Table: Aircraft Performance, Takeoff (Boeing 757-232 Series) PW 2037 Engine, 5° Flaps *(Continued)*

	Runway Length in Meters								
Weight	Reference Factor R								
(1000 kg)	60	70	80	90	100	110	120	130	140
70	1247	1438	1617	1786	1951	2114	2279	2451	2632
75	1409	1630	1843	2048	2249	2448	2646	2846	3051
80	1581	1838	2096	2350	2599	2837	3062	3271	3460
85	1769	2067	2377	2688	2990	3271	3522	3731	3888
90	1975	2319	2685	3056	3414	3742	4021	4234	4364
95	2205	2599	3022	3451	3864	4239	4554	4788	4917
100	2462	2912	3388	3868	4330	4754	5117		
105	2750	3261	3782	4301	4804	5276			
~~110~~	3074	3650	4207	4748					

18-3. THE FAA AIRPORT REFERENCE CODE

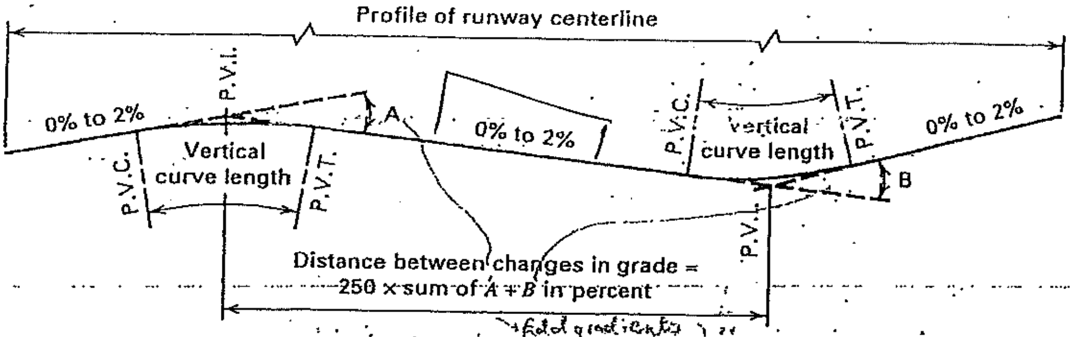
The FAA uses an airport reference code (ARC) system to relate airport design criteria to the operational and physical characteristics of the airplanes intended to operate at the airport [2]. The ARC has two components: the first depicted by a letter indicating the aircraft approach speed category; and the second, the airplane design group, depicted by a Roman numeral which depends on the critical airplane wingspan. Table 18-3 shows the determinants of the approach categories and the airplane design groups.

Table 18-3 Determinants of the FAA Airport Reference Code

FAA Aircraft Approach Category	
Approach Speed	Aircraft Approach Category
Speed less than 91 knots	A
Speed 91 knots or more but less than 121 knots	B 2
Speed 121 knots or more but less than 141 knots	C
Speed 141 knots or more but less than 166 knots	D 1,5
Speed 166 knots or more	E
FAA Airplane Design Groups	
Wingspan	Airplane Design Group (ADG)
Up to but not including 15 m (49 feet)	I
15 m (49 feet) up to but not including 24 m (79 feet)	II
24 m (79 feet) up to but not including 36 m (118 feet)	III
36 m (118 feet) up to but not including 52 m (171 feet)	IV
52 m (171 feet) up to but not including 65 m (214 feet)	V
65 m (214 feet) up to but not including 80 m (262 feet)	VI

Source: Airport Design, FAA Advisory Circular 150/5300-13, including Changes 1–4, Federal Aviation Administration, Washington, DC. September 29, 1989.

$T_F = 1.8\,T_C + 32$



Vertical curves

Length of vertical curves will not be less than 300′ for each 1% grade change, ~~except that no vertical curve will be required when grade change is less than 0.4%.~~

Grade change

Maximum grade change such as (*A*) or (*B*) should not exceed 2%.

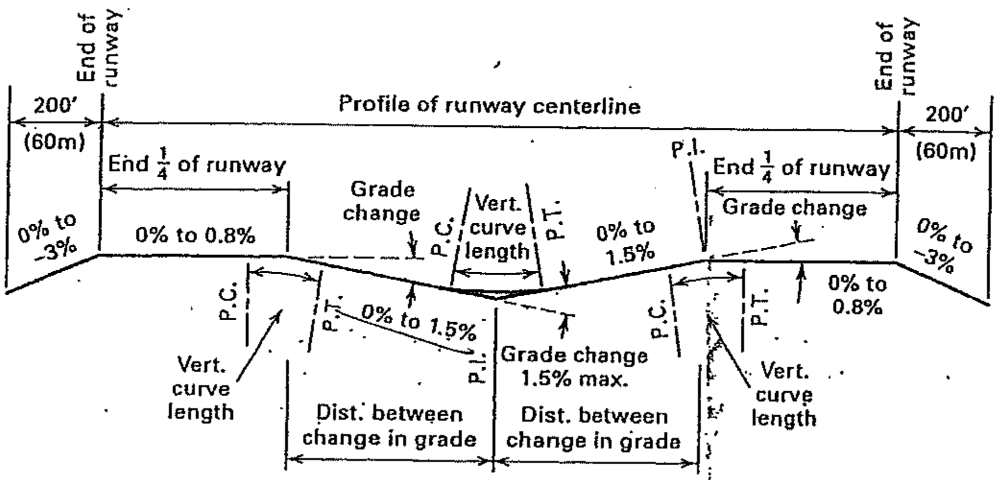
Figure 18-3a Longitudinal grade limitations for aircraft approach categories A and B. (Source: *Airport Design*, FAA Advisory Circular 150/5300-13, Changes 1–4, September 29, 1989.)

Generally, runway standards depend on aircraft approach speed, airplane wingspan, and the designated or planned approach visibility minimums. Taxiway and taxilane standards are related to airplane design group.

18-4. LONGITUDINAL GRADE DESIGN FOR RUNWAYS AND TAXIWAYS

In the interests of safe and efficient aircraft operations, runway grades should be flat, and grade changes should be avoided. A maximum longitudinal grade of 1.5 percent generally is specified for airports with aircraft approach categories C and D and a 2.0 percent maximum grade is recommended for approach categories A and B [2].

Where grade changes are necessary, the recommended maximum changes in grade and minimum lengths of vertical curves given by Fig. 18-3*a* and *b* should be used. Careful



Minimum distance between change in grade = 1000′ (300m) · sum of grade changes (in percent).
Minimum length of vertical curves = 1000′ (300m) · grade change (in percent).

Figure 18-3b Longitudinal grade limitations for aircraft approach categories C and D. (Source: *Airport Design*, FAA Advisory Circular 150/5300-13, Changes 1–4, September 29, 1989.)

thought should be given to what the future runway gradient requirements may be, and the runway gradient design should provide for future needs as well as present needs.

To minimize any hazard associated with objects on the runway, sight distance should be provided, preferably for the full length of the runway. However, an acceptable runway profile permits any two points 1.5 m (5 ft) above the runway centerline to be mutually visible for the entire runway length. If the runway has a full-length parallel taxiway, the runway profile may be such that an unobstructed line of sight will exist from any point 1.5 m (5 ft) above the runway centerline for one-half of the runway length.

There are no line-of-sight requirements for taxiways. However, the sighted distance along a runway from an intersecting taxiway needs to be sufficient to allow a taxiing aircraft to enter safely or cross the runway.

18-5. RUNWAY AND TAXIWAY CROSS SECTION

Although much larger in scale, a runway cross section resembles that of a highway. The runway, a paved, load-bearing roadway, is typically 18 m (60 ft) wide at small airports and 45 m (150 ft) wide at large airports.

Graded border areas are provided along each side of the runway as a safety measure should an aircraft lose control and veer from the runway. The border areas, which are typically stabilized earth with grass cover, vary in width from 7.5 m (25 ft) at the smallest airports to 53 m (175 ft) at the largest airports. The runway with adjacent borders is called the *runway safety area.* The width of the runway safety area varies from 36 to 150 m (120 to 500 ft), depending on the airport class. It extends a minimum of 72 m (200 ft) beyond each runway end.

The FAA uses the term *shoulder* to designate a relatively narrow paved or otherwise treated area adjacent to a runway or taxiway to resist jet erosion and/or to accommodate maintenance equipment.

The taxiway structural pavement is typically 7.5 to 15 m (25 to 50 ft) wide at smaller airports and 15 to 30 m (50 to 100 ft) at air carrier airports. In the latter case, an additional 4.6 m (15 ft) of pavement width should be provided on taxiway curves. In the interests of safety, taxiway centerlines are located 45 to 183 m (150 to 600 ft) from runway centerlines.

A typical runway and taxiway cross section is shown in Figs. 18-4*a* and *b.* Tables 18-4a and b give recommended runway dimensional standards for aircraft approach categories A and B, while similar standards for aircraft approach categories B and C are given in Tables 18-5 [3]. Taxiway dimensional standards are shown in Table 18-6.

18-6. TAXIWAYS AND TURNAROUNDS

Taxiways are used to facilitate the movement of aircraft to and from the runways. Where air traffic warrants, the usual procedure is to provide a taxiway parallel to the runway centerline for the entire length of the runway. This makes it possible for landing aircraft to exit the runway more quickly and decreases delays to other aircraft waiting to use the runway.

At smaller airports, air traffic may not be sufficient to justify the construction of a parallel taxiway. In this case, taxiing is done on the runway itself and turnarounds should be constructed at the ends of the runways. A typical taxiway turnaround is shown in Fig. 18-5. When construction of a full parallel taxiway is not practicable, a partial parallel taxiway may be suitable.

The design of the taxiway system will be determined by the volume of air traffic, the runway configuration, and the location of the terminal building and other ground facilities. The following general guidelines should be helpful in designing the taxiway system:

1. Taxiway routes should be direct and uncomplicated. Generally, taxiways should follow straight lines, and curves of long radius should be used when curves are required.

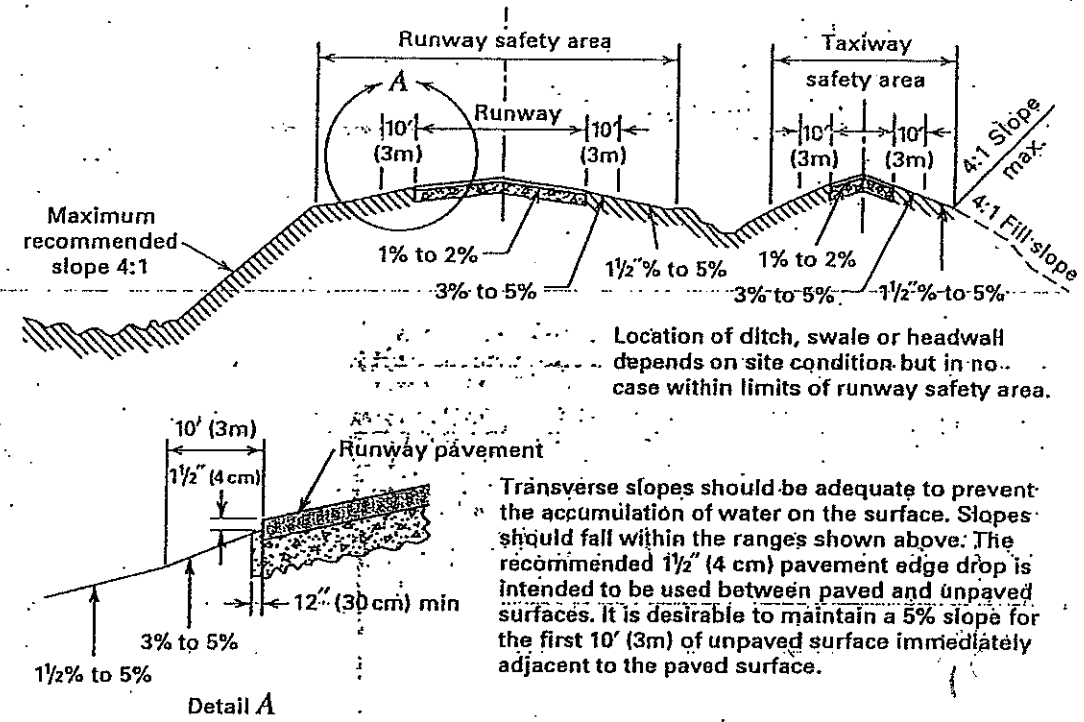


Figure 18-4a Transverse grade limitations for aircraft approach categories A and B. (Source: *Airport Design*, FAA Advisory Circular 150/5300-13, Changes 1–4, September 29, 1989.)

2. Whenever possible, taxiways should be designed so as not to cross active runways or other taxiways.
3. A sufficient number of taxiways should be provided in order to avoid congestion and complicated routes between runway exit points and the apron area.

At large and busy airports, the time an average aircraft occupies the runway frequently will determine the capacity of the runway system and the airport as a whole. This indi-

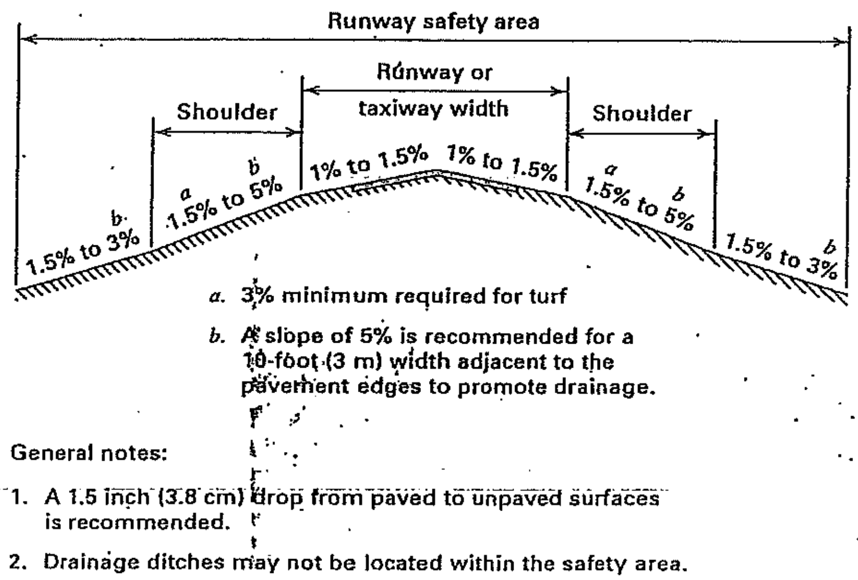


Figure 18-4b Transverse grade limitations for aircraft approach categories C and D. (Source: *Airport Design*, FAA Advisory Circular 150/5300-13, Changes 1–4, September 29, 1989.)

~~Table 18-4a Runway Design and Separation Standards for Aircraft Approach Category~~ A & B, Visual Runways and Runways With Not Lower Than 3/4 statute mile (1200 m) Approach Visibility Minimums

Item	I[a]	I	II	III	IV
	Airplane Design Group				
Runway Width	60 ft	60 ft	75 ft	100 ft	150 ft
	18 m	18 m	23 m	30 m	45 m
Runway Shoulder Width	10 ft	10 ft	10 ft	20 ft	25 ft
	3 m	3 m	3 m	6 m	7.5 m
Runway Blast Pad Width	80 ft	80 ft	95 ft	140 ft	200 ft
	24 m	24 m	29 m	42 m	60 m
Runway Blast Pad Length	60 ft	100 ft	150 ft	200 ft	200 ft
	18 m	30 m	45 m	60 m	60 m
Runway Safety Area Width	120 ft	120 ft	150 ft	300 ft	500 ft
	36 m	36 m	45 m	90 m	150 m
Runway Safety Area Length Beyond RW End	240 ft	240 ft	300 ft	600 ft	1,000 ft
	72 m	72 m	90 m	180 m	300 m
Runway Object Free Area Width	250 ft	400 ft	500 ft	800 ft	800 ft
	75 m	120 m	150 m	240 m	240 m
Runway Object Free Area Length Beyond RW end	240 ft	240 ft	300 ft	600 ft	1,000 ft
	72 m	72 m	90 m	180 m	300 m
Runway Centerline to:					
Taxiway/Taxilane Centerline	150 ft	225 ft	240 ft	300 ft	400 ft
	45 m	67.5 m	72 m	90 m	120 m
Aircraft Parking Area	125 ft	200 ft	250 ft	400 ft	500 ft
	37.5m	60 m	75 m	102 m	150 m

[a]These dimensional standards pertain to facilities for small airplanes exclusively.

Source: Airport Design, Federal Aviation Administration Advisory Circular 150/5300-13, including Change 4, Washington, DC, September 20, 1989.

cates that exit taxiways should be located conveniently so that landing aircraft can vacate the runway as soon as possible.

At smaller airports, three exit taxiways will generally be sufficient: one at the center and one at each end of the runway.

Two common types of exit taxiways are illustrated in Fig. 18-6. Perpendicular exit taxiways may be used when the design peak-hour traffic is less than 30 operations per hour. To expedite the movement of landing aircraft from the runway, most modern air carrier airports provide exit taxiways that are oriented at an angle of about 30 degrees to the runway centerline. This makes it possible for aircraft to leave the runway at speeds up to 96 km/hr (60 mph), increasing the efficiency and capacity of the airport system.

The proper location of exit taxiways is dependent on the touchdown point and landing roll of the aircraft as well as the configurations of the exits. The FAA [2] recommends that the points of curvature (PCs) of the angled type of exit be located at intervals beginning approximately 914 m (3000 ft) from the threshold to approximately 610 m (2000 ft) of the stop end

Table 18-4b Runway Design and Separation Standards for Aircraft Approach Category A and B Runways With Lower Than 3/4-statute mile (1200 m) Approach Visibility Minimums

Item	Airplane Design Group				
	I[a]	I	II	III	IV
Runway Width	75 ft 23 m	100 ft 30 m	100 ft 30 m	100 ft 30 m	150 ft 45 m
Runway Shoulder Width	10 ft 3 m	10 ft 3 m	10 ft 3 m	20 ft 6 m	25 ft 7.5 m
Runway Blast Pad Width	95 ft 29 m	120 ft 36 m	120 ft 36 m	140 ft 42 m	200 ft 60 m
Runway Blast Pad Length	60 ft 18 m	100 ft 30 m	150 ft 45 m	200 ft 60 m	200 ft 60 m
Runway Safety Area Width	300 ft 90 m	300 ft 90 m	300 ft 90 m	400 ft 120 m	500 ft 150 m
Runway Safety Area Length Beyond RW End	600 ft 180 m	600 ft 180 m	600 ft 180 m	800 ft 240 m	1,000 ft 300 m
Runway Object Free Area Width	800 ft 240 m	800 ft 240 m	800 ft 240 m	800 ft 240 m	800 ft 240 m
Runway Object Free Area Length Beyond RW End	600 ft 180 m	600 ft 180 m	600 ft 180 m	800 ft 240 m	1,000 ft 300 m
Runway Centerline to:					
Taxiway/Taxilane Centerline	200 ft 60 m	250 ft 75 m	300 ft 90 m	350 ft 105 m	400 ft 120 m
Aircraft Parking Area	400 ft 120m	400 ft 120 m	400 ft 120 m	400 ft 120 m	500 ft 150 m

[a]These dimensional standards pertain to facilities for small airplanes exclusively.

Source: Airport Design, Federal Aviation Administration Advisory Circular 150/5300-13, including Change 4, Washington, DC, September 20, 1989.

of the runway. For the 90-degree type, the PCs of the taxiway exits should be located at intervals beginning about 1067 m (3500 ft) from the threshold to approximately 610 m (2000 ft) from the stop end of the runway. Where the runway length exceeds 2134 m (7000 ft), intermediate exits at intervals of approximately 457 m (1500 ft) are recommended.

18-7. HOLDING BAYS

A holding bay is an area provided adjacent to the taxiway near the runway entrance for aircraft to park briefly while the cockpit checks and engine runups are made preparatory to take-off. The use of holding bays reduces interference between departing aircraft and minimizes delays at this portion of the runway system.

In the case of smaller airports, the FAA [2] recommends that holding bays be installed when air activity reaches 30 operations per normal peak hour. Space to accommodate at least two but not more than four aircraft is recommended for small airports.

Approximate amounts of holding bay space can be determined by applying factors to the wing spans of aircraft that will be using the apron. To determine the diameter of the

~~Table 18-5 Runway Design and Separation Standards for Aircraft Approach Categories~~ C & D

Item	Airplane Design Group I	II	III	IV	V	VI
Runway Width	100 ft 30 m	100 ft 30 m	100 ft 30 m	150 ft 45 m	150 ft 45 m	200 ft 60 m
Runway Shoulder Width	10 ft 3 m	10 ft 3 m	20 ft 6 m	25 ft 7.5 m	35 ft 10.5 m	40 ft 12 m
Runway Blast Pad Width	120 ft 36 m	120 ft 36 m	140 ft 42 m	200 ft 60 m	220 ft 66 m	280 ft 84 m
Runway Blast Pad Length	100 ft 30 m	150 ft 45 m	200 ft 60 m	200 ft 60 m	400 ft 120 m	400 ft 120 m
Runway Safety Area Width	500 ft 150 m	500 ft 150 m	500 ft 150 m	500 ft 150 m	500 ft 150 m	500 ft 150 m
Runway Safety Area Length Beyond RW End	1,000 ft 300 m	1,000 ft 300 m	1,000 ft 300 m	1,000 ft 300 m	1,000 ft 300 m	1,000 ft 300 m
Runway Object Free Area Width	800 ft 240 m	800 ft 240 m	800 ft 240 m	800 ft 240 m	800 ft 240 m	800 ft 240 m
Runway Object Free Area Length Beyond RW End	1,000 ft 300 m	1,000 ft 300 m	1,000 ft 300 m	1,000 ft 300 m	1,000 ft 300 m	1,000 ft 300 m
Visual runways and runways with not lower than 3/4-statute mile (1200 m) approach visibility minimums						
Runway Centerline to:						
Taxiway/Taxilane Centerline	300 ft 90 m	300 ft 90 m	400 ft 120 m	400 ft 120 m	a a	600 ft 180 m
Aircraft Parking Area	400 ft 120 m	400 ft 120 m	500 ft 150 m	500 ft 150 m	500 ft 150 m	500 ft 150 m
Runways with lower than 3/4 statute mile (1200 m) approach visibility minimums						
Runway Centerline to:						
Taxiway/Taxilane Centerline	400 ft 120 m	400 ft 120 m	400 ft 120 m	400 ft 120 m	a a	600 ft 180 m
Aircraft Parking Area	500 ft 150 m	500 ft 150 m	500 ft 150 m	500 ft 150 m	500 ft 150 m	500 ft 150 m

[a]Varies with airport elevation. See source.

Source: Airport Design, Federal Aviation Administration Advisory Circular 150/5300-13, including Change 4, Washington, DC, September 20, 1989.

space required to maneuver and provide wing-tip clearance, the factors given in Table 18-7 ~~should be multiplied by the aircraft wing span.~~

Sketches of a typical holding bay configurations are shown in Fig. 18-7.

But the figures in next 2 pages are in

~~18-8~~. AIRPORT DRAINAGE

A well-designed drainage system is an essential requirement for the efficient and safe operation of an airport. Inadequate drainage facilities not only will result in costly damages due to flooding but also may cause hazards to air operations and even result in the temporary closing of a runway or airport.

The design of a drainage system is based on the fundamental principles of open channel flow given in Chapter 14 and, in certain respects, the design procedures for airport drainage are identical to those for railway and highway drainage. The computation of runoff, for example, is accomplished by the rational formula, described in Chapter 14. On

Table 18-6 Taxiway Dimensional and Separation Standards

Item	Airplane Design Group I	II	III	IV	V	VI
Taxiway Width	25 ft 7.5 m	35 ft 10.5 m	50 ft[a] 15 m[a]	75 ft 23 m	75 ft 23 m	100 ft 30 m
Taxiway Edge Safety Margin[b]	5 ft 1.5 m	7.5 ft 2.25 m	10 ft[c] 3 m[c]	15 ft 4.5 m	15 ft 4.5 m	20 ft 6 m
Taxiway Shoulder Width	10 ft 3 m	10 ft 3 m	20 ft 6 m	25 ft 7.5 m	35 ft[d] 10.5 m[d]	40 ft[d] 12 m[d]
Taxiway Safety Area Width	49 ft 15 m	79 ft 24 m	118 ft 36 m	171 ft 52 m	214 ft 65 m	262 ft 80 m
Taxiway Object Free Area Width	89 ft 27 m	131 ft 40 m	186 ft 57 m	259 ft 79 m	320 ft 97 m	386 ft 118 m
Taxilane Object Free Area Width	79 ft 24 m	115 ft 35 m	162 ft 49 m	225 ft 68 m	276 ft 84 m	334 ft 102 m
Taxiway Centerline to:						
Parallel Taxiway/ Taxilane Centerline	69 ft 21 m	105 ft 32 m	152 ft 46.5 m	215 ft 65.5 m	267 ft 81 m	324 ft 99 m
Fixed or Movable Object	44.5 ft 13.5 m	65.5 ft 20 m	93 ft 28.5 m	129.5 ft 39.5 m	160 ft 48.5 m	193 ft 59 m
Taxilane Centerline to:						
Parallel Taxilane Centerline	64 ft 19.5 m	97 ft 29.5 m	140 ft 42.5 m	198 ft 60 m	245 ft 74.5 m	298 ft 91 m
Fixed or Movable Object	39.5 ft 12 m	57.5 ft 17.5 m	81 ft 24.5 m	112.5 ft 34 m	138 ft 42 m	167 ft 51 m

[a]For airplanes in Airplane Design Group III with a wheelbase equal to or greater than 60 feet (18 m), the standard taxiway width is 60 feet (18 m).

[b]The taxiway edge safety margin is the minimum acceptable distance between the outside of the airplane wheels and the pavement edge.

[c]For airplanes in Airplane Design Group III with a wheelbase equal to or greater than 60 feet (18 m), the taxiway edge safety margin is 15 feet (4.5 m).

[d]Airplanes in Airplane Design Groups V and VI normally require stabilized or paved taxiway shoulder surfaces.

Source: Airport Design, Federal Aviation Administration Advisory Circular 150/5300-13, including Change 4, Washington, DC, September 20, 1989.

the other hand, an airport has certain peculiarities regarding its drainage requirements. Characterized by extensive areas and flat slopes and a critical need for the prompt removal of surface and subsurface water, airports usually are provided with an integrated drainage system. This system consists of surface ditches, inlets, and an underground storm drainage system. Typical drainage systems are shown in reference 3.

The underground conduits are designed to operate with open channel flow, and because pipe sections in this system are long, uniform flow can be assumed. The hydraulic design of the channels and conduits, therefore, usually is accomplished by application of the Manning equation.

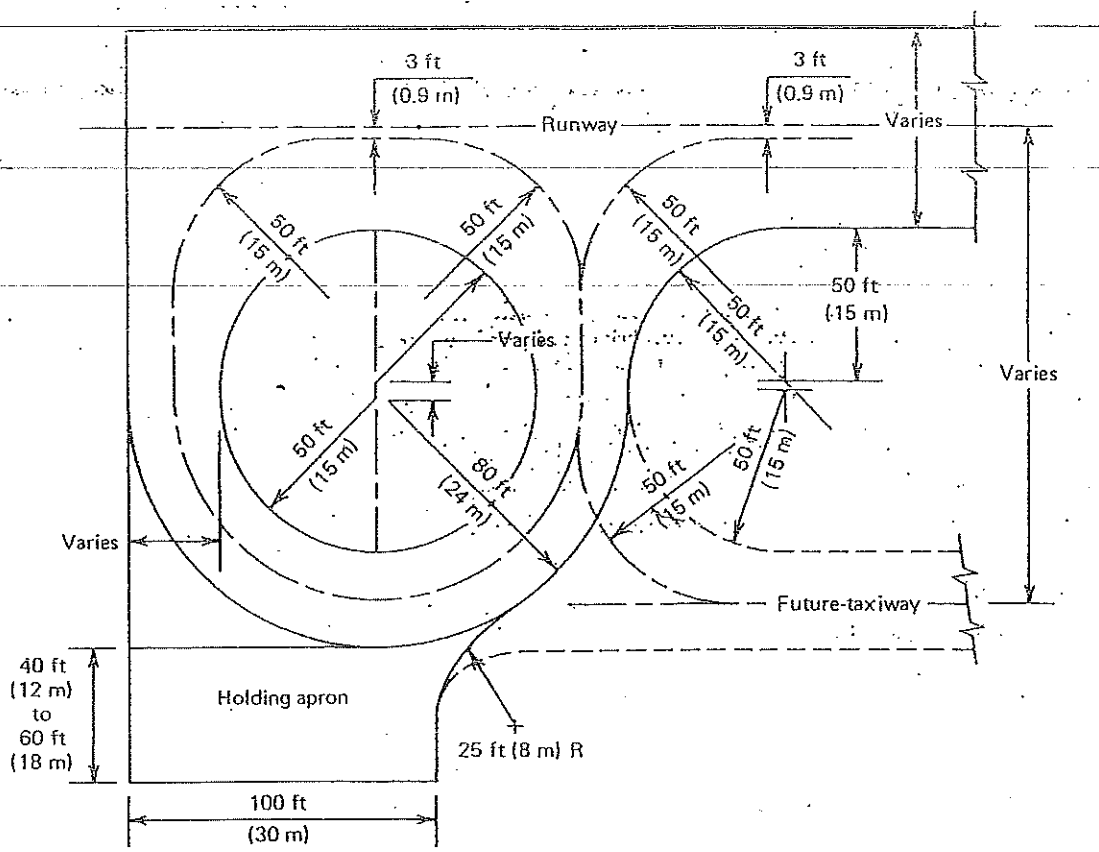


Figure 18-5 Circular taxiway turnaround for a smaller airport. (Courtesy Federal Aviation Administration.)

Storm drain inlets are placed as needed at low points and are typically spaced at 300-to-500-ft intervals. Manholes are provided to permit workers to inspect and maintain the underground system. Manholes are commonly placed at every abrupt change of direction and approximately every 90 to 120 m (300 to 400 ft) on tangents. Typical inlets and manhole designs are given by reference 3.

The design of a drainage system for an airport involves the following steps:

1. Using the proposed grading plan as a basis, a layout of the drainage system is made. The grading plan, which should show the proposed finished grade by 1-ft contour lines, will make it possible to select appropriate locations for drainage ditches and inlets and to determine the tentative layout of the underground pipe system.
2. Drainage structures and pipelines usually are identified by numbers or letters for easy reference in design computations.
3. For each drainage subarea, the runoff is computed by means of a rational formula. (See Chapter 14.) This involves the estimation of a runoff coefficient and a time of concentration (including flow time in the pipe system) and the selection of a design rainfall intensity from an intensity–duration curve similar to Fig. 14-1. In this connection, the FAA recommends a storm frequency of 5 years [3].
4. Beginning with the uppermost pipe section, the slope and pipe size are selected to carry the design flow. Design charts such as that shown in Fig. 18-8 are used for this purpose. As the design progresses along the line, each succeeding pipe section carries the water from its surface drainage area plus that contributed through its inlet structure.

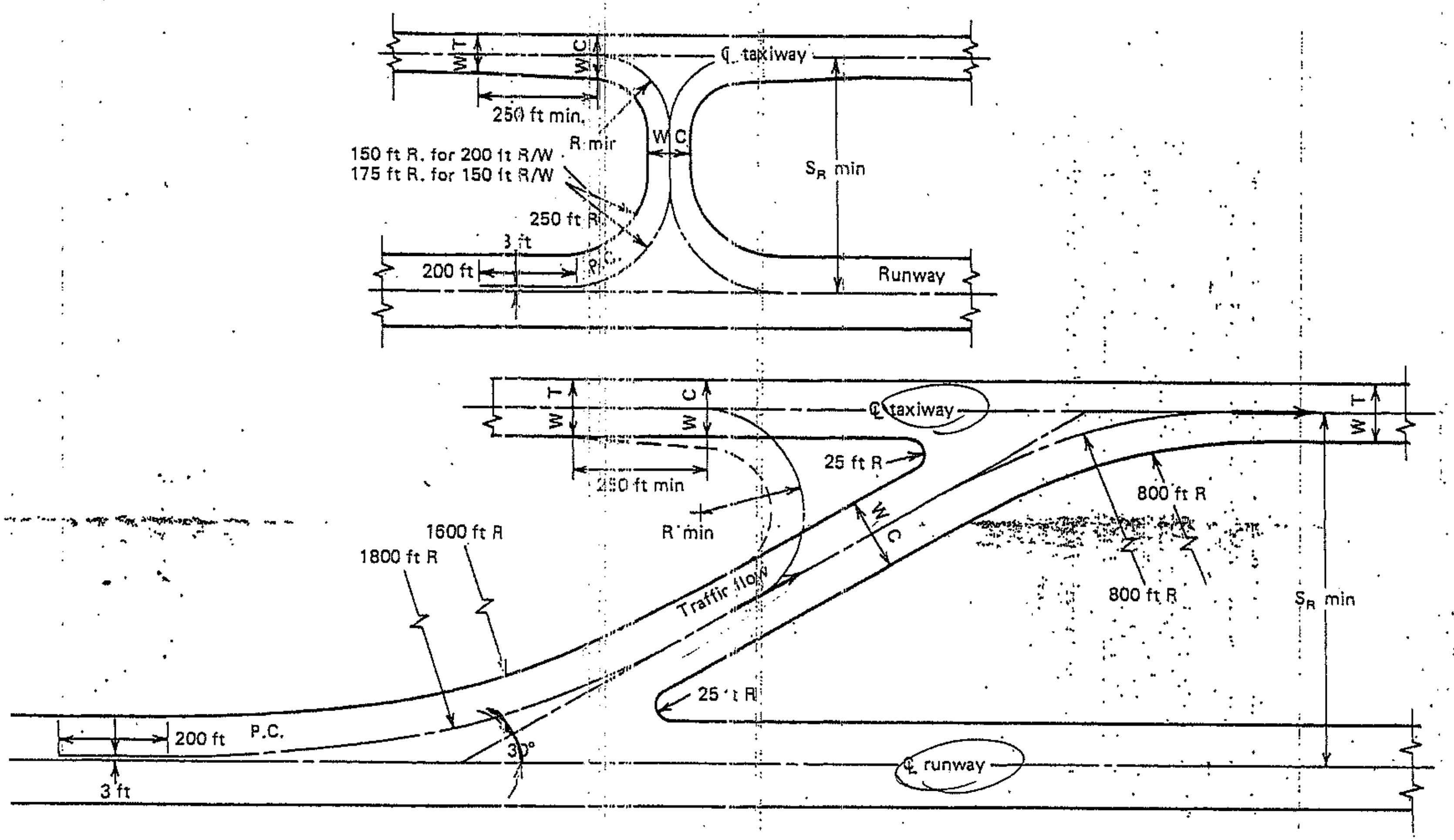


Figure 18-6 Recommended design for exit taxiways. (Courtesy Federal Aviation Administration.)

Table 18-7 Factors for Determination of Holding Apron Space

Aircraft Type	Factor
Aircraft with single-wheel gear	1.50–1.65
Aircraft with dual-wheel undercarriages	1.35–1.50
Aircraft with dual-tandem gear	1.60–1.75

Example problems for the actual design of a drainage system for a portion of an airport have been abstracted from the FAA publication *Airport Drainage* [3] and are given here.

EXAMPLE 18-4 ***Drainage Design without Ponding***

Suppose it is desired to design an underground drainage system to accommodate the surface flow from the apron and taxiways shown by Fig. 18-9. Inlets and line segments are first numbered and lengths are scaled from the map and recorded as shown by columns 1, 2, and 3 in Table 18-8.

Columns 4 through 10 record the data required for the calculation of runoff for various subareas in the system. These calculations are made by the rational formula, which is described adequately in Chapter 14. It is noted that for a given inlet, time of concentration equals the inlet time (Column 4), or time required for water to flow overland from the most remote point in the subarea, plus flow time (column 5) through the particular pipe segments. Flow time is computed by dividing the pipe length by the velocity of flow (column 12).

Column 11 shows the accumulated runoff that must be accommodated.

Columns 12 through 16 show data pertaining to the hydraulic design of the system. The slope of a pipe section (column 14) is based on such factors as topography, amount of cover, depth of excavation, elevation of the discharge basin or channel, and discharge velocity. With the slope and accumulated runoff, the size of pipe required (column 13), velocity of flow (column 12), and pipe capacity (column 15) can be determined by means of a design chart for the Manning equation similar to Fig. 18-8. In this example, concrete pipe was used and a Manning roughness coefficient, *n*, of 0.015 was assumed.

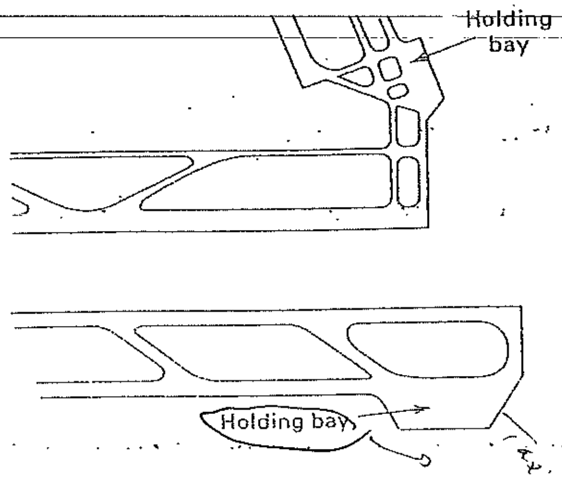


Figure 18-7 Typical holding bay configurations.

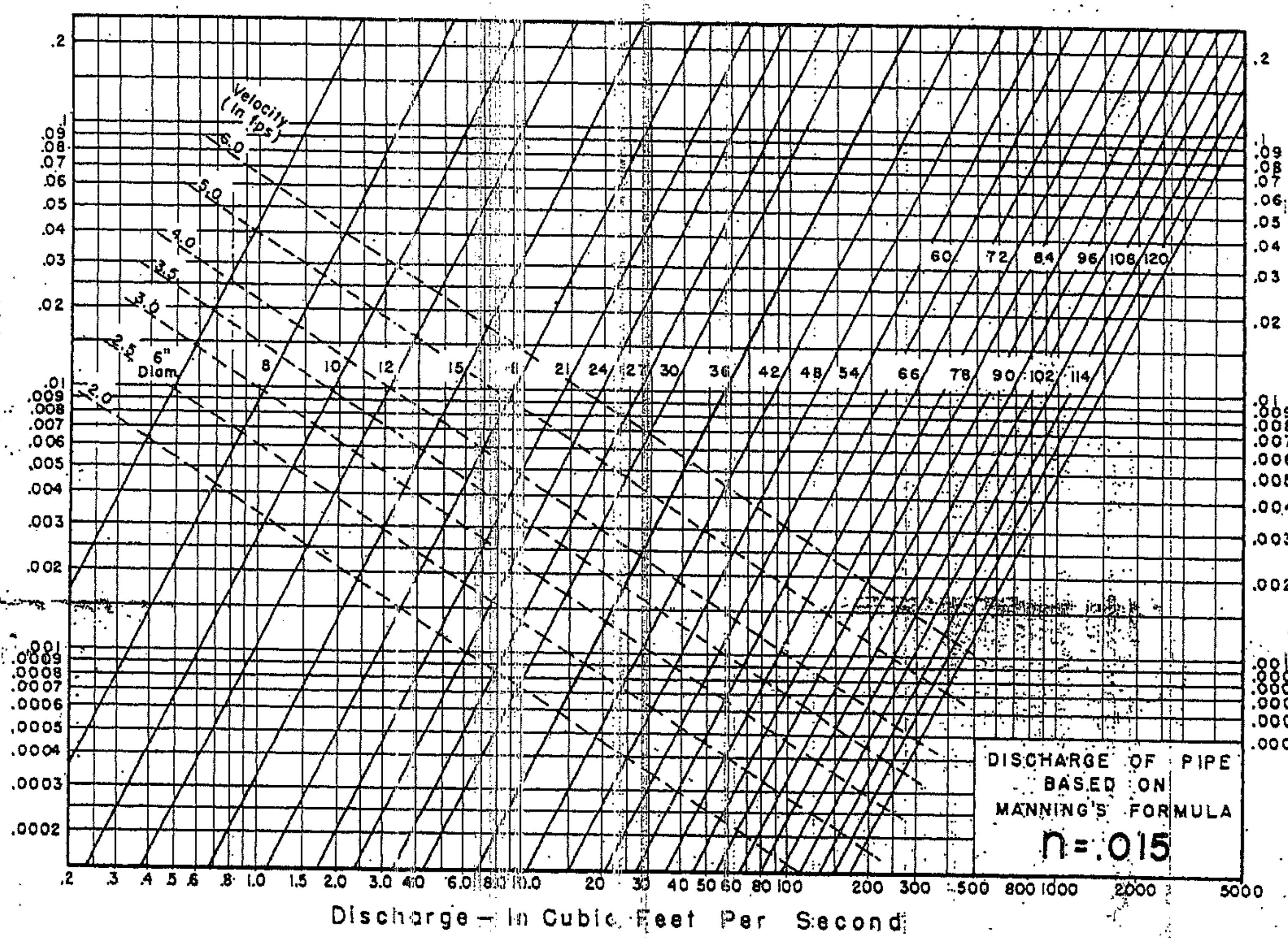


Figure 18-8 Design chart for uniform flow. (Source: *Airport Drainage*, FAA Advisory Circular 150/5320-5B, July 1, 1970.)

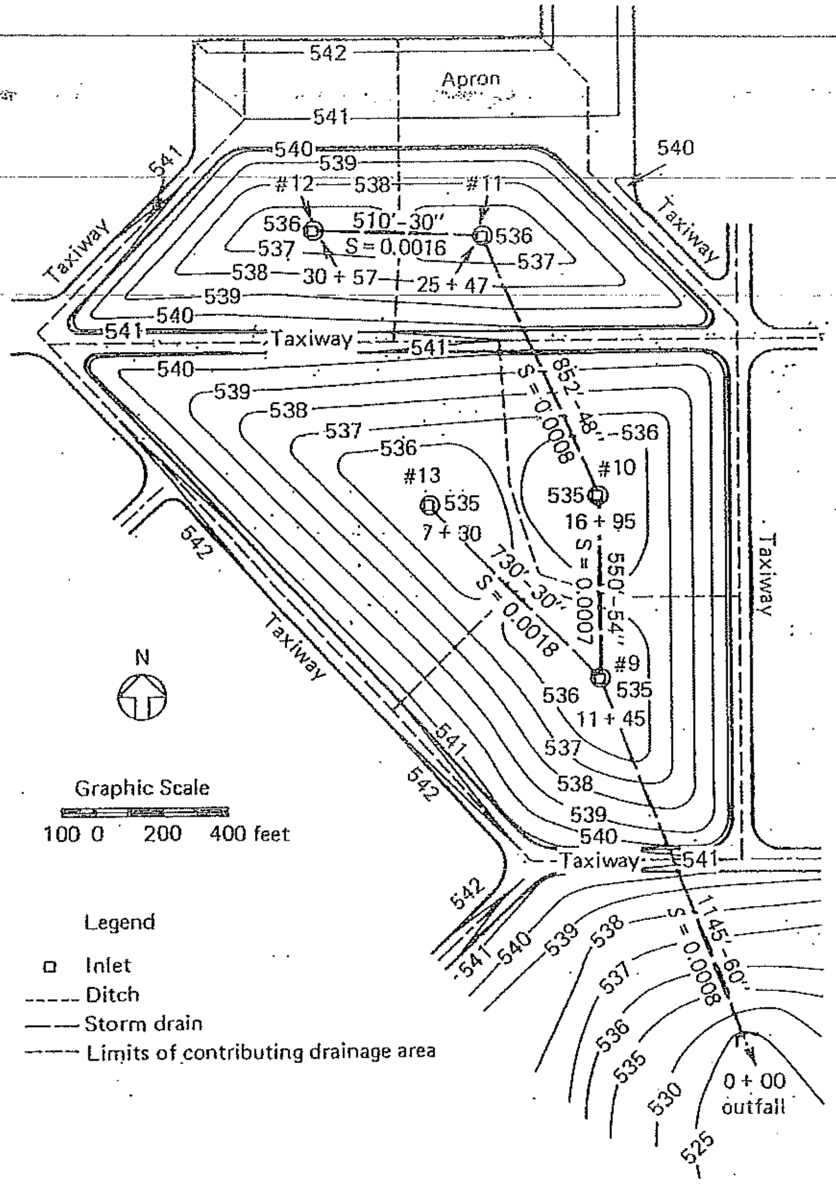


Figure 18-9 Portion of an airport showing drainage design. (Source: *Airport Drainage*, FAA Advisory Circular 150/5320-5B, July 1, 1970.)

EXAMPLE 18-5[2] *Drainage Design with Ponding*

Suppose we wish to drain the area shown in Fig. 18-10 with a single pipe to permit ponding of a short duration between the taxiways. It will be noted that this area is part of that shown in Fig. 18-9, except the contours have been changed to permit drainage by a single inlet. Suppose further that a 24-in. pipe is used to drain the area and that this pipe is to be installed on a 0.7 percent slope. Given the data shown below, what would be the maximum ponding expected within a 5-year period?

Runoff coefficient = 0.354

Drainage area = 49.52 acres

[2]Conventional U.S. units are used in this example to be consistent with the available design chart shown in Fig. 18-8.

Table 18-8 Drainage System Design Data for Example 18-4

(1) Inlet	(2) Line Segment	(3) Length of Segment (ft)	(4) Inlet Time (min)	(5) Flow Time (min)	(6) Time of Concentration (min)	(7) Runoff Coefficient, C	(8) Rainfall Intensity, I (in./hr)	(9) Tributary Area, A (acres)
12	12–11	510	52	3.4	52	0.49	1.98	14.69
11	11–10	852	53	5.0	55.4	0.53	1.90	14.72
10	10–9	550	39	3.3	60.4	0.35	1.78	11.97
13	13–9	730	62	3.9	62	0.35	1.76	21.50
9	9–out	1145	42	4.2	65.9	0.35	1.70	16.05
Out								

Note: Time of concentration for inlet 11 is 55.4 min (52 + 3.4 = 55.4), which is the most time remote point for this inlet. Likewise time of concentration for inlet 10 is 60.4 min (52 + 3.4 + 5.0 = 60.4).

Source: Airport Drainage, FAA Advisory Circular AC 150/5320-5B, July 1, 1970.

From Fig. 18-7, it will be noted that the discharge for this pipe will be about 15.9 ft^3/sec. The runoff that can be accommodated by this pipe is a linear function of time and is plotted in Fig. 18-11.

Based on the rational formula, the amount of runoff (ft^3/sec) is

$$Q = CIA$$

$$= 0.354 \times I \times 49.52$$

The rainfall intensity, I, which is dependent on duration, can be obtained for various durations from Fig. 14-2. In a 10-min period, for example, one would expect, for a 5-year flood frequency, a maximum runoff of

$$Q = 0.354 \times 4.68 \times 49.52 = 82.04 \text{ ft}^3/\text{sec}$$

Thus, the corresponding runoff value in cubic feet would be

$$\text{Runoff} = 82.04 \times 600 \text{ sec} = 49{,}224 \text{ ft}^3$$

Similarly, for a 30-min period,

$$Q = 0.354 \times 2.74 \times 49.52 = 48.03 \text{ ft}^3/\text{sec}$$

$$\text{Runoff} = 48.03 \times 1800 \text{ sec} = 86{,}485 \text{ ft}^3$$

Runoff values have been computed for other times and are plotted in Fig. 18-11. It will be noted from the graph that the maximum difference between the 24-in. pipe capacity line and the cumulative runoff curve occurs at a time of approximately 60 min and that the maximum ponding value is

$$P = 53{,}830 \text{ ft}^3$$

While this value is less than the storage capacity between the inlet and contour 536,

(10) Runoff, Q (ft^3/sec)	(11) Accumulated Runoff (ft^3/sec)	(12) Velocity of Drain (ft/sec)	(13) Size of Pipe (in.)	(14) Slope of Pipe (ft/ft)	(15) Capacity of Pipe (ft^3/sec)	(16) Invert Elevation	(17) Remarks
14.25	14.25	2.8	30	0.0016	14.25	530.96	(n = 0.015)
14.82	29.07	2.8	48	0.0008	35.0	528.64	See note below.
7.46	36.53	2.8	54	0.0007	45.0	527.46	See note below.
13.24	13.24	3.1	30	0.0018	15.0	530.38	
9.55	59.32	3.3	60	0.0008	65.0	526.57	
						525.65	

Fig. 18-11 indicates that it would require more than 2 hr for the 24-in. pipe to empty the ponding area even when considering a 5-year flood. Since ponding over a long period of time is undesirable from the standpoint of safety and pavement performance, a larger culvert should be used. Fig. 18-11 shows that if a 30-in. pipe were used, ponding would occur for only slightly more than 1 hr.

18-9. GRADING AND EARTHWORK

The proper grading of an airport is required to provide safe and efficient grades for aircraft operations to maintain good surface drainage and to control erosion. Airport grading is characterized by wide, flat, and rounded slopes, with smooth transitions provided between graded and ungraded areas. Yet, because of the costs associated with earthwork operations, unnecessary grading should be avoided. Where future expansion of an airport is anticipated, the grading should be consistent with the ultimate proposed grades. Proposed grading operations usually are shown by means of a grading plan that shows original and proposed contour lines. (See Fig. 18-12.)

Grading quantities are usually computed by the average end-area formula discussed in Chapter 14. See Eqs. 14-13 and 14-14. However, because of the relatively flat topography and large expanses of areas to be graded at airports, it may be advantageous to consider the areas enclosed by the original and final contour lines as end areas, A_1 and A_2. These areas can be measured with a planimeter and the contour interval becomes the length, L. Where an embankment or excavation section ends between two contours, the vertical distance must be estimated.

AIRPORT MARKING AND LIGHTING

18-10. VISUAL AIDS REQUIREMENTS

During the major portion of flights while flying at altitude, pilots are assisted by magnetic compasses, gyros, and electronic devices. However, most instruments are not reliable when the aircraft is within about 60 m (200 ft) of the ground. Thus, landings and take-offs are accomplished largely "by eye," and during these critical operations various visual aids are placed at an airport to assist pilots. For operations in the daytime and in good weather, pilots are aided by airport markings. In inclement weather and at night they depend on airport lighting.

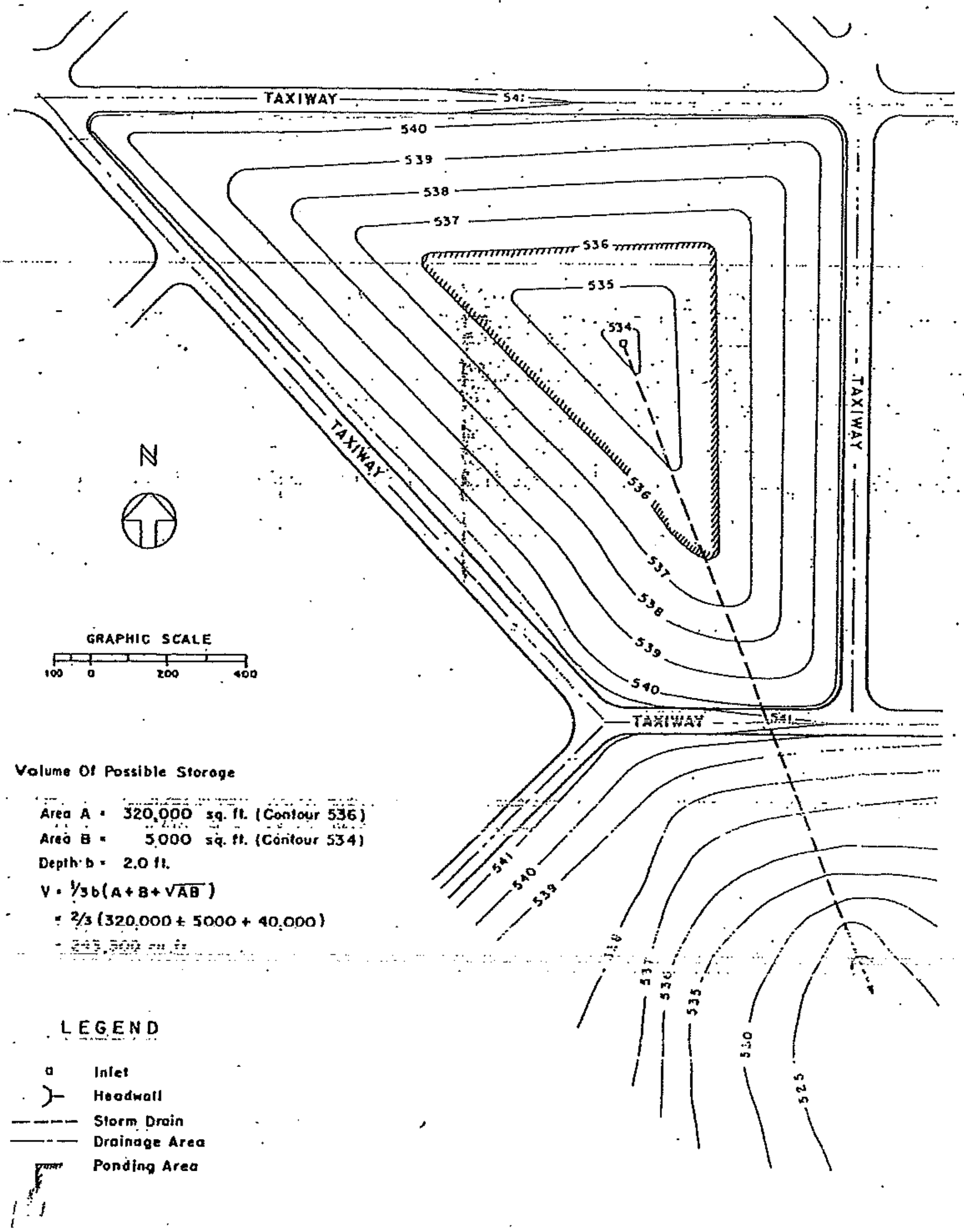


Figure 18-10 Example of providing for ponding area. (Source: *Airport Drainage*, FAA Advisory Circular 150/5320-5B, July 1, 1970.)

In addition to visual aids to help pilots locate and identify an airport or runway, they especially need assistance in properly approaching the runway and landing the aircraft. Walter and Roggenveen [4] have pointed out that, while landing, an airplane is a moving coordinate system that is approaching a stationary coordinate system, the runway. These coordinate systems are shown in Fig. 18-13. The aircraft not only may move about each of the three axes, but it also may rotate about them. Therefore, the pilot must rotate, orient, and translate the aircraft so that it coincides with the coordinate system of the runway.

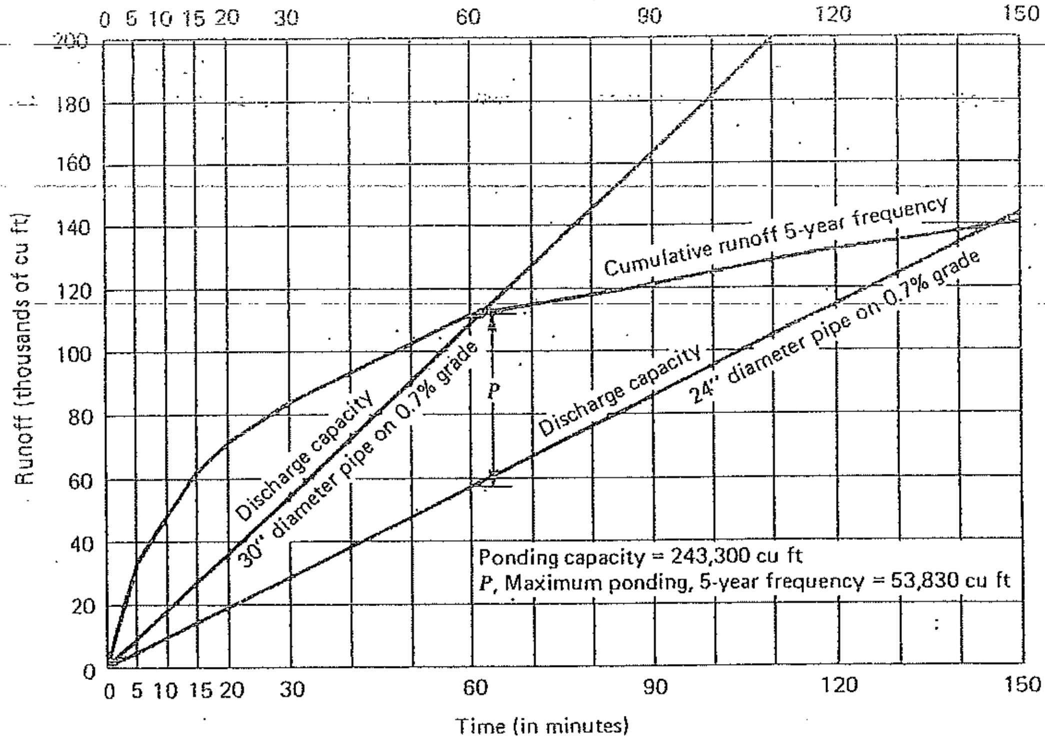


Figure 18-11 Cumulative runoff for ponding in Fig. 18-10. (Source: Adapted from *Airport Drainage,* FAA Advisory Circular 150/5320-5B, July 1, 1970.)

To make a safe landing, a pilot must make correct judgments regarding:

1. Alignment—whether or not the plane is headed straight for the runway
2. Roll—whether or not the aircraft is banked properly in relation to the ground surface
3. The height of the aircraft above the runway
4. Its distance from the end of the runway

In periods of good visibility these judgments can be made by reference to familiar objects on the ground such as trees and buildings. When the visibility is restricted due to inclement weather or darkness, the pilot requires visual aids in the form of airport markings and lights.

18-11. HANGAR AND STRIP MARKERS

At small airports that do not have paved runways, the landing strip may be identified by strip markings such as that shown in Fig. 18-14. For additional locational guidance, the name of the airport and an arrow showing true north may be provided as a hangar marker.

18-12. THE SEGMENTED CIRCLE MARKER SYSTEM

Another visual aid commonly used at small airports is the segmented circle. It consists of a series of pointed markers arranged in the form of a circle of 15 m (50 ft) radius. The segments of the circle are typically 0.9 m (3 ft) in horizontal width and 1.8 to 3.6 m (6 to 12 ft) in length. Typical details are shown in Fig. 18-15.

The segmented circle helps a pilot to identify an airport and provides a standard location for various signal devices. A wind cone or sock usually is placed at the center of the

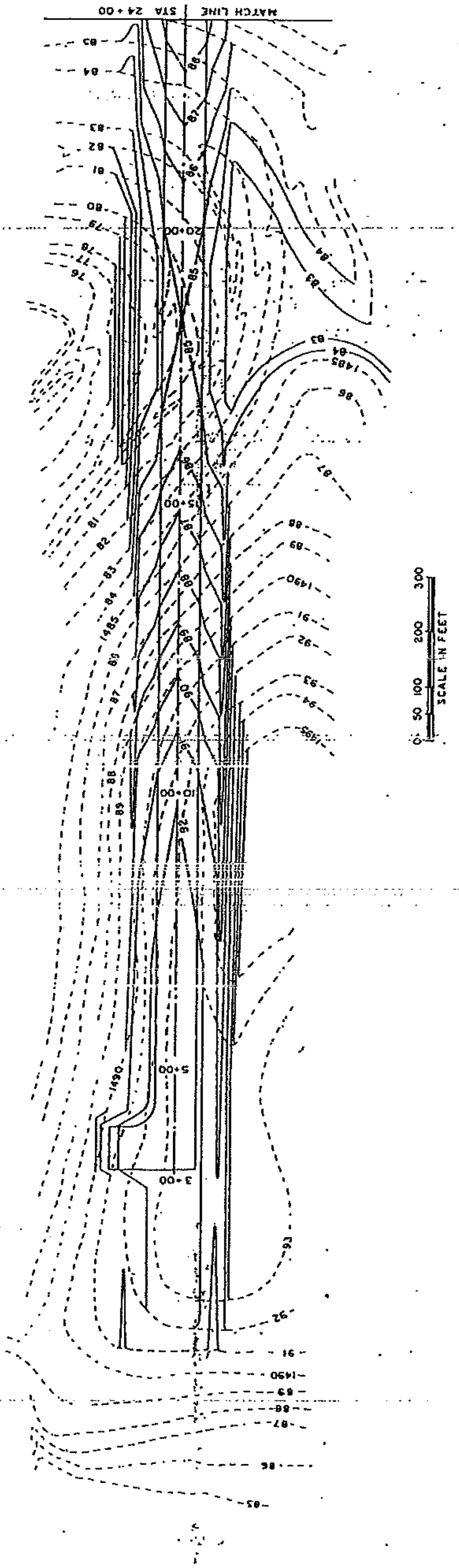


Figure 18-12 A typical grading plan. (Courtesy Federal Aviation Administration.)

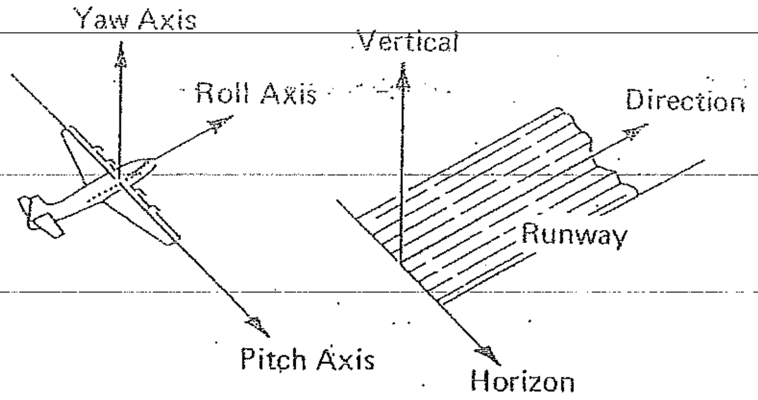


Figure 18-13 Runway and airplane coordinate systems. (Source: Walter and Roggeveen, "Airport Approach, Runway and Taxiway Lighting Systems," *Journal of the Air Transport Division*, ASCE, June 1958.)

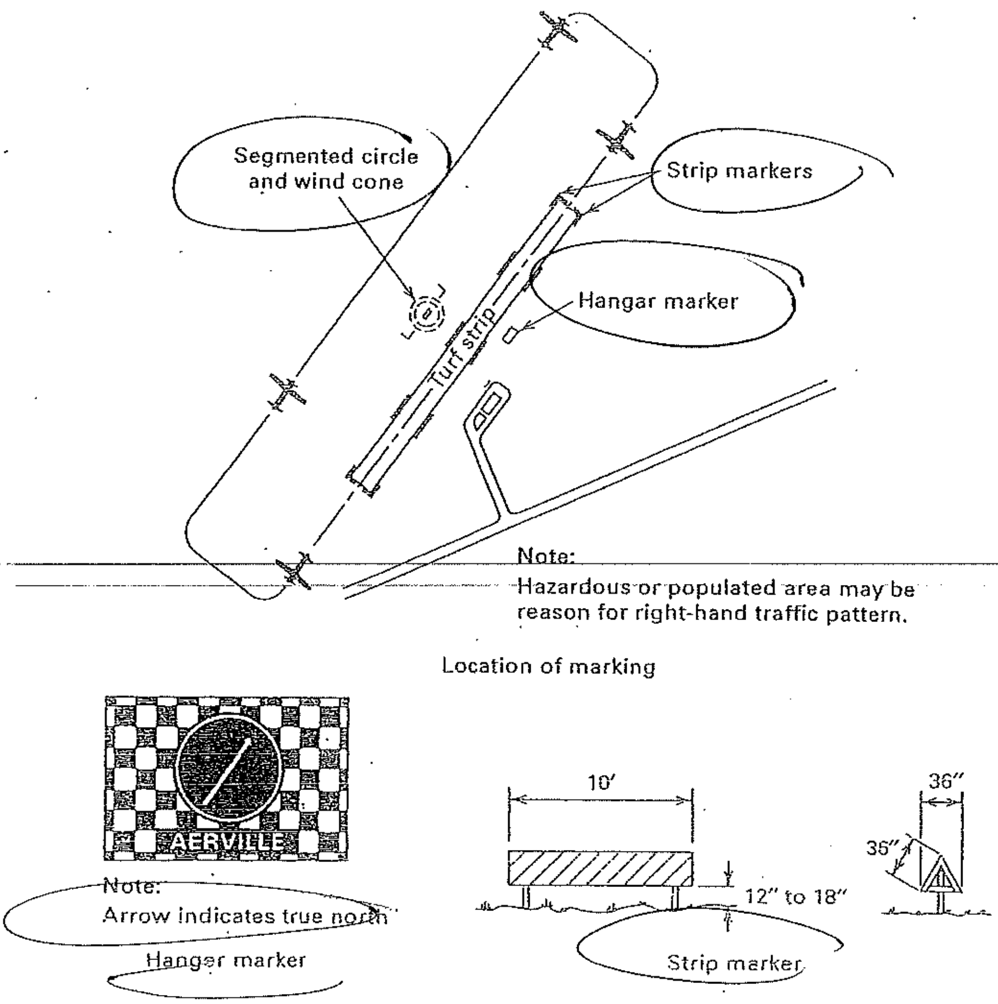


Figure 18-14 Hanger and strip markers for small airports. (Courtesy Federal Aviation Administration.)

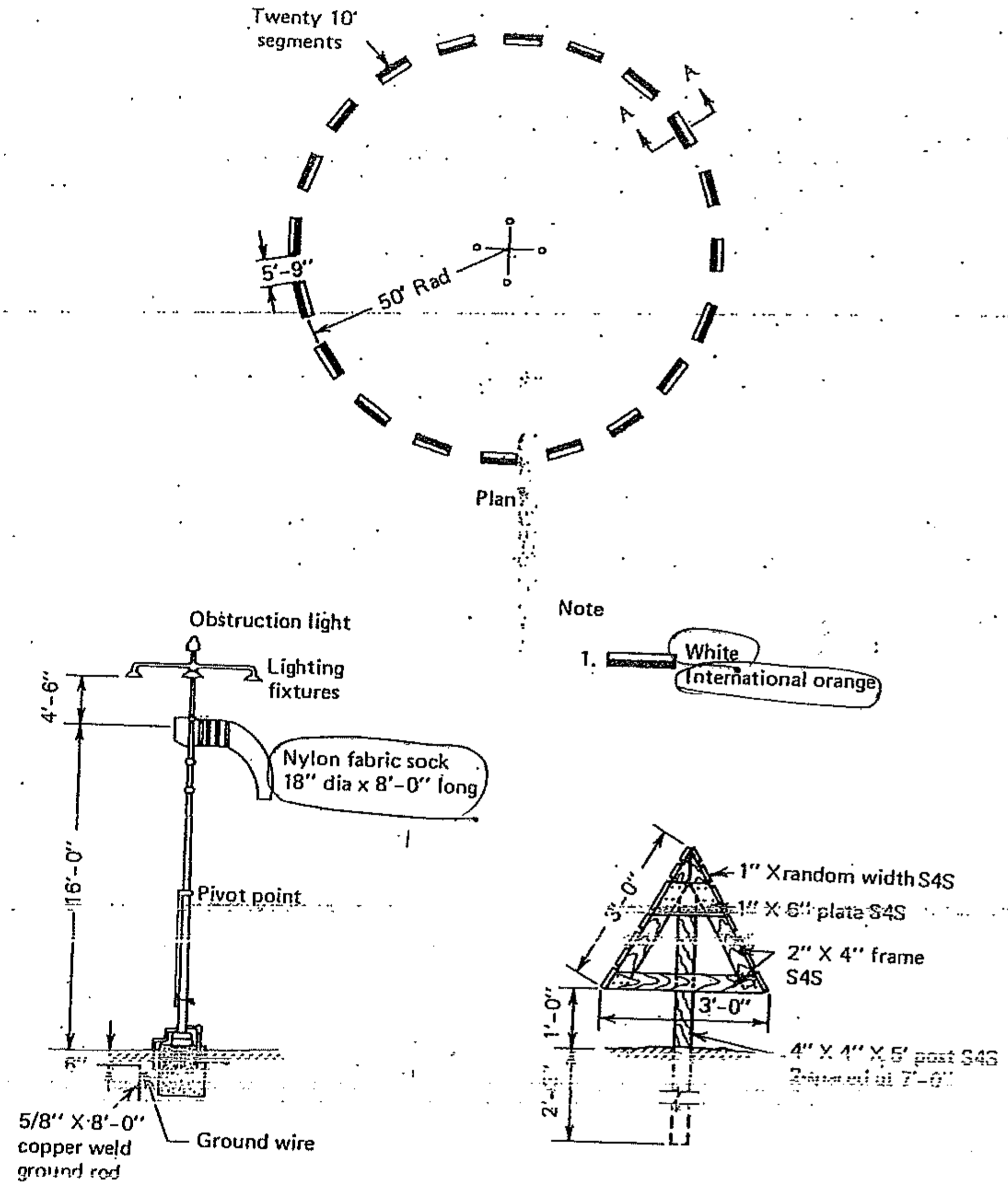


Figure 18-15 Segmented circle airport marker system. (Courtesy Federal Aviation Administration.)

segmented circle. If the airport has more than one runway, a landing direction indicator may be provided at the center of the circle in the form of an arrow or tee. To indicate the landing pattern and orientation of landing strips, indicators may be placed at the periphery of the segmented circle.

18-13. RUNWAY AND TAXIWAY MARKING

Runway and taxiway marking consists of numbers and stripes that are painted on the pavement. Each end of a runway is marked with a number nearest one-tenth the magnetic azimuth of the runway centerline measured clockwise from the magnetic north. For example, a runway oriented N 10°E would be numbered 1 on the south end and 19 on the north end. Additional information is needed when two or more parallel runways are used, and the designations L, C, and R are used to identify left, center, and right runways, respectively. Where four or five parallel runways are used, two of the runways are assigned numbers of the next nearest one-tenth of the magnetic azimuth. The numbers and letters are about 18 m (60 ft) tall and 6 m (20 ft) wide.

For purposes of runway marking, the FAA groups runways into three classes: (1) visual runway, (2) nonprecision instrument runway, and (3) precision instrument runway. Recommended markings for these runway classes are shown in Fig. 18-16. In addition to runway numbers, the following runway markings may be used:

1. A dashed centered line stripe
2. Threshold markers
3. Side stripes
4. Markings indicating distance from the end of the runway

The latter group of markings, which are used only on precision instrument runways, include fixed-distance markings placed 300 m (1000 ft) from each runway end; touchdown zone markings placed 150 m (500 ft) from each runway end; and additional markings provided at 150-m (500-ft) intervals to indicate location with respect to the end of the runway. (See Fig. 18-16.) Runway markings are usually white but may be outlined in black to make them more conspicuous.

Taxiway markings consist of a continuous centerline stripe and holding lines offset at least 30 m (100 ft) from the runway edge. Taxiway markings are normally yellow.

18-14. AIRPORT LIGHTING

The lighting that should be provided for a given airport depends on the airport size, the nature and volume of air traffic at night and during periods of inclement weather, and local meteorological conditions. Five types of airport lighting are described in the following paragraphs:

1. Obstruction lighting
2. Airport beacons
3. Approach lighting
4. Runway lighting
5. Taxiway lighting

This listing is not intended to be an exhaustive one, but it includes those major classes of airport lighting utilized to facilitate aircraft operations at night and in inclement weather. Illumination for wind cones and wind tees, ceiling light projectors, and lighting for aprons, hangars, and auto parking lots, though important, will not be described here due to limitations in space.

18-15. OBSTRUCTION LIGHTING

In the interests of air safety, obstruction lights must be placed on towers, bridges, smokestacks, and other structures that may constitute a hazard to air navigation. Single and double obstruction lights, flashing beacons, and rotating beacons are used to warn pilots during darkness or others periods of limited visibility of the presence of obstructions. A standard color, aviation red, is used for these lights.

The number, type, and placement of obstruction lights on a given structure will depend principally on its height. The FAA standards for the lighting of obstructions are given in the publication *Obstruction Marking and Lighting* [5].

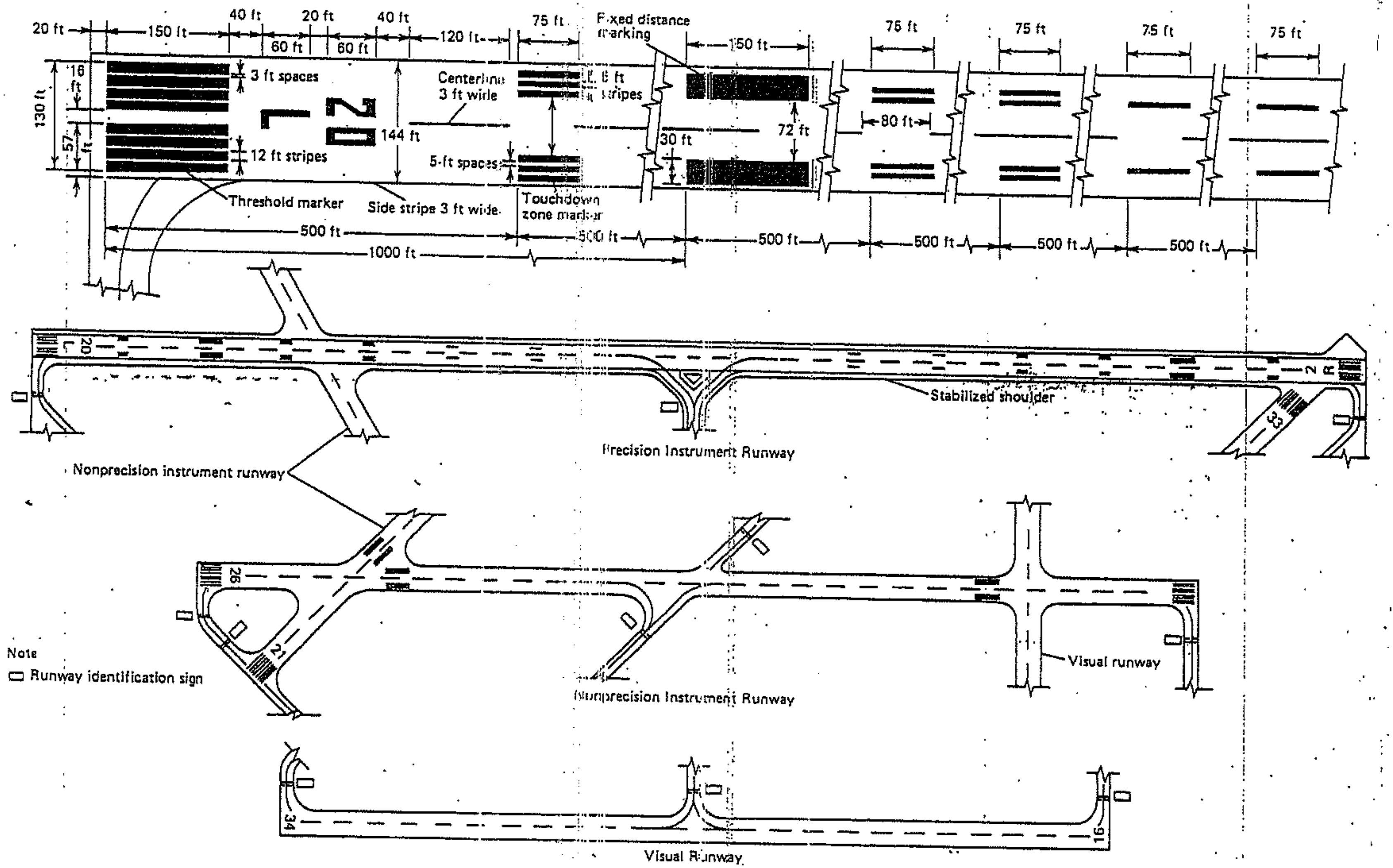


Figure 18-16 Runway and taxiway marking. (Source: Reference 9.)

18-16. AIRPORT BEACONS

The location and presence of an airport at night is indicated by an airport beacon. While limited use has been made of a 250-mm (10-in.) "junior" beacon, a 414-mm (36-in.) beacon typically is used. The standard airport beacon rotates at a speed of 6 rpm and is equipped with an optical system that projects two beams of light 180 degrees apart. One light beam is green and the other is clear. A split beam is used to distinguish military airports.

18-17. APPROACH LIGHTING

Approach lights provide guidance to pilots during the few seconds it takes to travel from the approach area to the runway threshold. A pilot must be able to immediately identify and easily interpret approach lights. These lights provide the pilot with information regarding the alignment, roll, and height of the aircraft and its distance from the runway threshold.

The basic needs for approach lights have been determined objectively and thoughtfully on the basis of the glide angle, visual range, cockpit cutoff angle, and landing speed of the aircraft (taken to be 1.3 times the stalling speed). It is agreed, for example, that approach lights should extend at least 427 m (1400 ft) from the runway threshold for nonprecision approaches and about 915 m (3000 ft) for precision approaches. However, the FAA only relatively recently has standardized approach lighting requirements, and a confusing variety of approach lighting systems remain in use. The FAA recommends four standard approach lighting systems. These systems can be grouped into two categories:

1. Medium-intensity systems
2. High-intensity systems

All of the recommended FAA approach lighting configurations have a series of light bars installed perpendicular to the extended runway centerline at specified spacings. These light bars are composed of five lamps each. All of the recommended FAA configurations also feature a wide light bar installed at a distance of 300 m (1000 ft) from the runway threshold. Sketches of FAA approach lighting systems are shown in Fig. 18-17. Except as noted, approach lights are white.

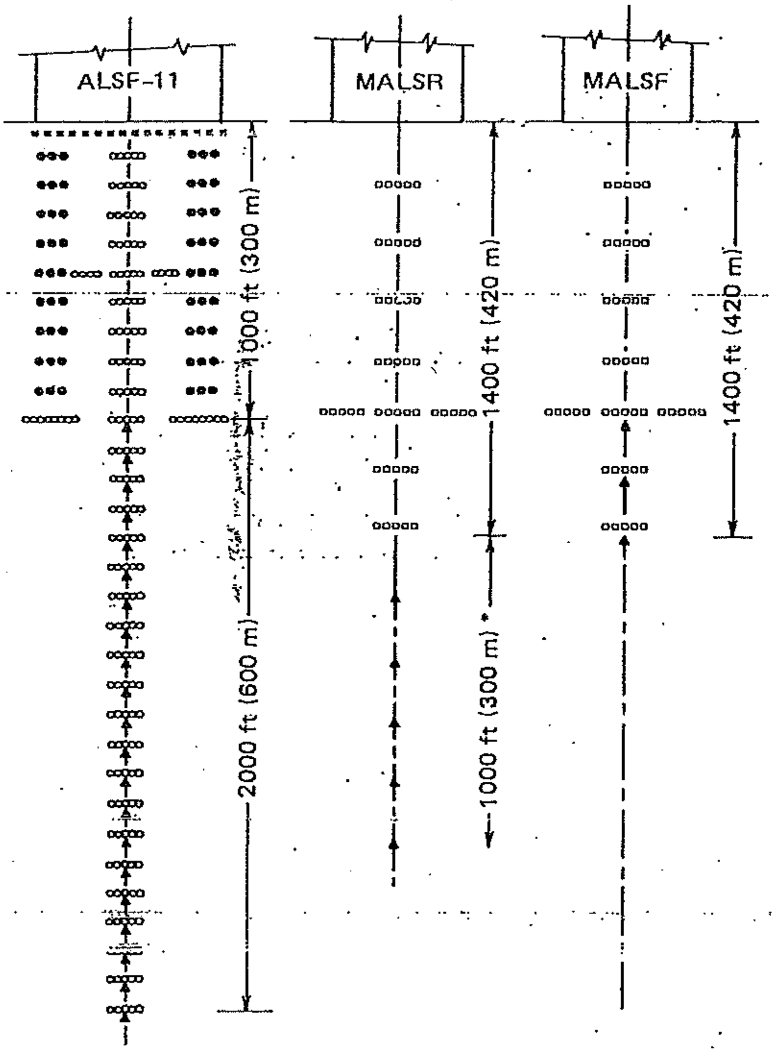
Medium-Intensity Systems. Medium-intensity approach lighting systems are economy systems that utilize 150-W lamps. These systems are recommended for utility airports. There are three recommended medium-intensity approach configurations:

MALS—medium-intensity approach lighting system.

MALSF—medium-intensity approach lighting system with sequenced flashers. This is the same as the MALS except it is equipped with three sequenced flasher lights. This system would be used where approach area identification problems exist.

MALSR—medium-intensity lighting with runway alignment indicator lights. This system is similar to the MALS except that five flashing lights are installed along the extended runway centerline at 61-m (200-ft) spacings extending the total length of 732 m (2400 ft). Where the glide slope angle is less than 2.75 degrees, eight flashing lights are recommended, providing a total length of 414 m (3000 ft).

High-Intensity Systems. These elaborate and expensive approach lighting systems are used most commonly at air carrier airports to facilitate ILS approaches. In addition to steady burning 300-W lights, these configurations feature a system of sequenced flashing



High-intensity steady burning white lights.
Medium-intensity steady burning white lights.
Steady burning red lights.
Sequenced flashing lights.
ALS threshold light bar.

Figure 18-17 FAA approach lighting systems. (Source: Reference 10.)

lights. One such light is installed at each centerline bar starting 300 m (1000 ft) from the threshold and extending outward to the end of the system. The sequenced flashing lights appear as a ball of light traveling at a high speed.

The ALSF-II configuration was designed for operations in very poor visibility, Category II or lower. It is distinguished by two wide bands of barrettes or red lights located on either side of the centerline along the inner 300 m (1000 ft) of the approach area. This configuration has a crossbar of white lights at 150 m (500 ft) from the threshold in addition to the standard bar at 300 m (1000 ft). (See Fig. 18-17.)

18-18. OTHER VISUAL AIDS FOR AIRCRAFT APPROACHES

Additional visual aids systems used to facilitate easier and safer aircraft approaches are (1) VASI, (2) PAPI, and (3) REIL.

The Visual Approach Slope Indicator System (VASI). This basically consists of light bars placed on each side of the runway 183 m (600 ft) from the runway end (downwind bars) and a second set of bars on each side of the runway 346 m (1300 ft) from the runway end (upwind bars). There are five basic VASI configurations, the 16-box, 12-box, 6-box, 4-box, and 2-box.

Each light box in the VASI system projects a split beam of light, the upper segment being white and the lower red. When a pilot makes an approach, he or she sees white lights if the approach is too high and red lights if the approach is too low. When a proper approach is made, the downwind bars appear white and the upwind bars appear red. This system is primarily intended for use in VFR weather conditions.

Precision Approach Path Indicator (PAPI). This two-color system consists of two or four identical light units in a row located on the left of the runway, perpendicular to the centerline. It is easily sited, set, and maintained and capable of multipath interpretation. On approach the pilot will see the signal display shown in Fig. 18-18, indicating his position. The PAPI systems will eventually replace VASI systems.

Runway End Identifier Lights (REIL). These sometimes are placed at the ends of runways to provide rapid and positive identification of the approach end of a runway. The system consists of two synchronized flashing lights, one on each end of the runway threshold, the beams of which are aimed 10 to 15 degrees outside a line parallel to the centerline. The REIL system is used where there is a preponderance of confusing lights from off-airport sources such as motels, automobile lights, and so on. It normally would not be used if sequenced flashers are used in the approach lighting system.

18-19. THRESHOLD AND RUNWAY LIGHTING

Three classes of lights are installed in or near the runway to aid pilots during the final approach and to facilitate landings, rollouts, and take-offs: (1) threshold lights; (2) runway edge lights; and (3) runway centerline and touchdown zone lights. Threshold lights, which consist of a line of green lights extending across the width of the runway, identify to the pilot the runway end and help him or her decide whether to complete the landing or to execute a missed approach.

Runway edge lights consist of lights installed not more than 3 m (10 ft) from the pave-

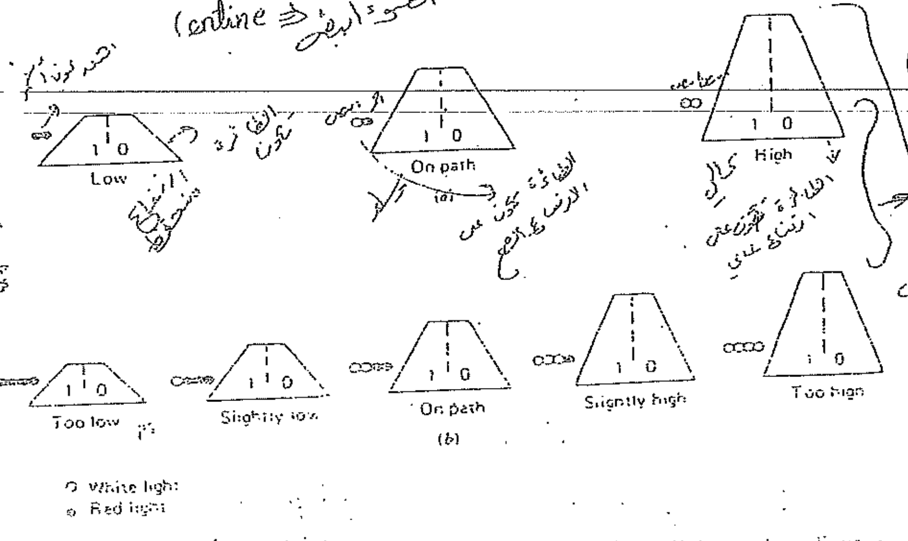


Figure 18-18 PAPI signal display. (a) Two-light system. (b) Four-light system. (Source: [illegible] Advisory Circular [illegible], May 23, 1985.)

Online

approach light

ment edge and spaced not more than 60 m (200 ft) on centers. These lights are aviation white except that aviation yellow is used in the last 610 m (2000 ft) of an instrument runway to indicate a caution zone.

Runway centerline and touchdown lighting systems are used in conjunction with electronic precision aids and high-intensity approach lighting systems under periods of limited visibility. These lights are built into the pavement surface. Runway centerline lights usually are installed at 15.2-m (50-ft) centers along the runway centerline (offset a maximum of 2 ft to clear centerline marking). These lights are white except for the end 914-m (3000-ft) sections of the runway.

In the zone from 914 to 305 m (3000 to 1000 ft) to runway end, alternately red and white lights are displayed to a departing pilot. In the final 305 m (1000 ft), all red lights are displayed along the centerline. Runway centerline lights are bidirectional, so that red lights in the 914-m (3000-ft) end zones appears as white lights to pilots approaching to land.

Touchdown zone lights consist of rows of transverse bars of white lights mounted flush in the pavement along 30-m (100-ft) centers. Such lights are employed in conjunction with high-intensity approach lighting systems to provide pilots additional roll guidance along a 914-m (3000-ft) length beyond the runway end.

18-20. TAXIWAY LIGHTING

Taxiway lighting provides guidance for pilots for maneuvering along the system of taxiways that connect the runways and the terminal and hangar aprons. The conventional taxiway lighting consists of omnidirectional blue lights located on each side of the taxiway pavement. These lights are offset not more than 3 m (10 ft) from the pavement edge and spaced longitudinally not more than 60 m (200 ft) apart. Much shorter spacings are required on short curves and at intersections.

Instead of taxiway edge lights, taxiway centerline lights may be installed in new construction and may supplement taxiway edge lights where operations occur in low visibility or where taxiing confusion exists.

The taxiway centerline lighting system consists of single semiflush lights with less than 9.5-mm ($\frac{3}{8}$-in.) protrusions above the pavement surface inset in the taxiway pavement along the centerline. These lights are steady burning and have a standard color of aviation green.

Detailed information on the design, installation, testing, maintenance, and inspection of taxiway edge and centerline systems are given, respectively, by references 6 and 7.

18-21. AIRPORT PAVING

It is regretted that space limitations preclude the inclusion of a discussion of the subject of airport paving. The reader is referred to the FAA publication *Airport Paving* [8] for a complete discussion of this important topic.

PROBLEMS

1. What length of runway is required for a smaller airport that is 6000 ft above sea level and has a normal maximum temperature of 75°F? The effective gradient is 1.2 percent. The airport must accommodate small airplanes having more than 10 seats.
2. What length of runway is required for a Boeing 757 Series 232 aircraft given the following design conditions?

Normal maximum temperature	75°F	
Airport elevation	4000 ft	(1219 m)
Take-off weight	220,000 lb	(99,873 kg)
Maximum landing weight	175,000 lb	(79,444 kg)
Effective runway gradient	0.85 percent	

3. Profile grade data for a proposed airport runway are given here. Does the proposed longitudinal grade design conform with the requirements of the FAA? The first vertical curve is 1600 ft long and the second is 1000 ft long. Use the Airplane Design Group VI and Approach Category D.

Station	Grade (Percent)	Comment
0 + 00		Begin runway
	−0.85	
30 + 50		PI station no. 1
	+0.65	
55 + 50		PI station no. 2
	−0.40	
92 + 00		End runway

4. Determine the following dimensions for an air carrier airport designed to accommodate a B-727-000 aircraft.
   a. Runway centerline to taxiway centerline
   b. Taxiway width on tangent
   c. Taxiway width on curve
   d. Radius of taxiway centerline on curves
5. Obtain rainfall data for your locality from the National Weather Service or from some other source. Prepare a rainfall intensity–duration curve similar to Fig. 14-1 (5-year curve). Using the other data given in Example 18-4, design a drainage system for the apron and taxiways shown in Fig. 18-9.
6. Design a drainage system for the apron and taxiways shown in Fig. 18-9 given the runoff values given here. Other required data are given by Table 18-8.

Inlet	Runoff, Q
12	29.4
11	23.1
10	10.7
13	17.4
9	14.8

REFERENCES

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2. *Airport Design*, FAA Advisory Circular 150/5300-13, Change 1-4, Federal Aviation Administration, Washington, DC, September 29, 1989.

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4. Walter, C. E., and V. J. Rogeveen, "Airport Approach, Runway and Taxiway Lighting Systems," *Journal of the Air Transport Division,* American Society of Civil Engineers, June 1978.
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8. *Airport Pavement Design and Evaluation,* FAA Advisory Circular 150/5320-6C, including Change 1 and 2, Federal Aviation Administration, Washington, DC, August 30, 1979.
9. *Marking Paved Areas at Airports,* FAA Advisory Circular 150/5340-1F, Federal Aviation Administration, Washington, DC, October 22, 1987.
10. *Airport Design Standards–Site Requirements for Terminal Navigation Facilities,* FAA Advisory Circular 150/5300-2D, including Change 1, Federal Aviation Administration, Washington, DC, 1980.

Part VI

Design of Water Transportation Facilities

The final part of this book is concerned with the layout and design of the infrastructure for water transportation: the harbors, the channels, and the ports. This part begins with a discussion of the nature of water movements and describes special problems in designing for the coastal environment. It lists desirable features for harbor sites and describes various structures used to construct artificial harbors and to protect coastal areas. The book then discusses various aspects of the planning, layout, and design of the aprons, buildings, and other components of modern ports.

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Chapter 19

Introduction to Water Transportation

Container :- 20 equivalent unit ~~Fet~~ feu

19-1. INTRODUCTION

Since ancient times, water transportation has broadened humanity's horizons and has influenced profoundly the growth and development of civilization. Historians report that as early as 6000 B.C. the Egyptians had ships with masts and sails, and galleys were used on the Nile River as early as 3000 B.C. During the reign of King Solomon (circa 961–922 B.C.), Phoenician galleys sailed from Biblical Tyre and Sidon bringing copper from Cyprus, papyrus from Egypt, and ivory, gold, and slaves from Africa. These large, low, and typically one-decked vessels were propelled by both sails and oars. Galleys, which often were manned by slaves, were used by Rome in her war with Carthage (A.D. 140). These vessels continued to be used throughout the Middle Ages, especially in the Mediterranean Sea.

weight :

In early America [1]: limit for container : volume :-

River boats were nothing more than huge rafts called flatboats *or broad-horns. Generally, they were flat-bottomed and boxlike, covered from stem to stern. The flatboat was a one-way vessel, dependent entirely upon currents for propulsion with only occasional guidance from its handlers. At the end of its downstream run, it was usually broken up and the lumber sold.*

The keelboat *began to appear on the rivers at about the turn of the nineteenth century. It was a long, narrow vessel with graceful lines, sturdily built to withstand many trips both downstream and upstream. The keelboat could carry as much as 80 tons of freight. It was floated downstream under careful guidance, and cordelled upstream. Cordelling took two forms: the crew walked along the bank and pulled the keelboat with ropes; or they literally pushed it upstream with iron-tipped poles which extended to the bottom of the river. One historian has estimated that there were as many as 500 keelboats operating on the Ohio River and its tributaries by 1819.*

The steamboat *was invented in 1807; and in 1811 the river steamer* New Orleans *was launch in Pittsburgh and went into operation between there and New Orleans. By 1835 New Orleans was posting the arrival of over 1000 steamboats per year. Records indicate that by 1852 the public landing at Cincinnati was recording the arrival of steamboats at an annual rate of 8,000, about one per hour. The tonnage handled by steamboats on the rivers of the United States at* *the height of the packet boat era, just before the War Between the States, is reported to have exceeded the tonnage handled by all the vessels of the British Empire.*

Although the feasibility of steamboat travel in the open sea was demonstrated by Colonel John Stevens in 1809, sailing vessels continued to dominate ocean transportation until shortly before the Civil War.

Scheduled ocean travel was initiated on January 5, 1818, when the *James Monroe,* a 424-ton ship, sailed from New York to Liverpool. This packet or liner service, which gave uncommon services to passengers and fast movement of freight, was highly successful. Larger, faster, and more expensive ships were built, additional lines were organized, and the packet service was extended.

The first half of the nineteenth century was a prosperous period for shipping companies. The quest to sail "at a fast clip" resulted in the design and building of over 400 clipper ships during the period from 1846 to 1855. These vessels traveled at speeds that rival some of the faster commercial ships in service today. Despite the speed and classic beauty of these vessels, the clipper ships were soon replaced by steam ships, and following the Civil War, there was a sharp decrease[1] in shipping under the American flag. This decline in U.S. influence in ocean shipping is felt to the present day.

19-2. RECENT GROWTH IN WATER TRANSPORTATION

In recent years, substantial increases in water transportation activity have been noted. These increases are attributed to growing world population, the development of new products and new sources of raw materials, and general industrial growth, especially in the petroleum industry. Accompanying the increases in water transportation tonnage, larger ships have been built and innovative storage and loading facilities have been developed. These changes have created an ever-increasing need for the most modern and efficient port facilities.

Data furnished by the Maritime Administration, U.S. Department of Transportation, indicate that there has been substantial growth in the number and gross tonnage[2] of the merchant ships of the world. Gross tonnage has grown at a much greater rate than has the number of ships, reflecting the trend to larger ships.

Steady growth is also occurring in the amount and value of commercial cargo carried in U.S. oceanborne foreign trade, as the following numbers show [2]:

Year	Thousands of Metric Tons	Millions of Dollars
1993	884,393	501,445
1994	913,408	564,570
1995	967,244	617,556

[1]For all practical purposes the river fleet was destroyed during the Civil War and was not rebuilt. There was little progress in inland water transportation from the end of the Civil War until about 1920 [1].

[2]Gross tonnage is a cubic measurement: one gross ton equals 100 ft^3 of storage space. The data given are for ocean-going ships of 1000 gross tons and over.

Somewhat less growth has been experienced in inland water transportation, as indicated by the data furnished by the U.S. Army Corps of Engineers [3]:

Year	Thousands of Short Tons
1992	1095
1993	1068
1994	1099

Today, nearly 99 percent of overseas freight tonnage is transported by ships, and approximately 15 percent of the freight (ton miles) within the continental United States moves on the inland waterway system.

19-3. THE NATURE OF WATER TRANSPORTATION

By its nature, water transportation is most suitable for bulky and heavy commodities that have to be moved long distances and for which time of transport is not a critical factor.

Table 19-1 lists the most common types of cargoes moved on U.S. oceanborne foreign trade routes by liner, nonliner, and tanker service. It is apparent from this table that liquid bulk cargo (especially petroleum and petroleum products) and dry bulk cargo (such as coal, cereal, and ores) are the dominant types of shipments.

Other common classes of waterborne cargo include machinery, fruits and vegetables, wood and timber, and motor vehicles and parts. In addition, a wide variety of manufactured commodities are shipped as packaged goods (general cargo) or in large sealed boxes called containers. (See Fig. 19-1.) The use of containers speeds dramatically the handling of certain types of freight while significantly decreasing shipping costs.

Some of the major commodities shipped on the inland waterways system are:

Petroleum and petroleum products
Coal and coke
Iron, steel, and aluminum ingots and plate
Scrap iron
Grains
Chemicals
Sugar, syrup, and molasses
Forest products and paper
Sand and gravel
Animals feeds and fertilizer

Inland waterway system

Freight movements on the Great Lakes consist predominantly of iron ore, steel, coal, and grain.

19-4. THE EFFECTS OF SHIP CHARACTERISTICS AND CARGO TYPE

The planning and design of port and harbor facilities is strongly dependent on the characteristics of the ships to be served and the types of cargo to be handled. The relevant characteristics of ships and the port characteristics they influence are [4]:

1. Main dimensions:

   a. Length, which governs the length and layout of single-berth terminals, the length of stretches of quay, and the location of transit sheds. The length also influences the widths and bends of channels and the size of port basins.

Table 19-1 Top Twelve Commodity Groups Shipped in 1994 by Liner, Nonliner, and Tanker Service

Rank	Commodity Group	Thousands of Metric Tons
	Liner Service	
1	Pulp of wood, waste paper	9,047
2	Nuclear reactors, boilers, machinery	7,329
3	Paper, paperboard, and articles	7,034
4	Vehicles, except railway or tramway; parts	6,525
5	Iron and steel	6,342
6	Plastics and articles thereof	6,290
7	Wood and articles of wood, and wood charcoal	5,756
8	Electric machinery, sound and TV equipment	3,626
9	Cereals	3,584
10	Beverages, spirits and vinegar	3,498
11	Edible fruit and nuts, fruit and melon peel	3,248
12	Organic chemicals	3,181
	Nonliner Service	
1	Mineral fuel, oil, bitumen, etc.	82,921
2	Cereals	68,381
3	Salt, sulfur, earth and stone, lime, etc.	42,810
4	Ores, slag and ash	32,055
5	Iron and steel	28,411
6	Wood, wood articles, wood charcoal	19,662
7	Oil seeds, miscellaneous grain, seed, fruit, etc.	16,357
8	Food industry residues, prepared animal feed	12,491
9	Inorganic chemicals, rare earth metal compounds, etc.	7,158
10	Edible fruit and nuts, fruit and melon peel	4,452
11	Fertilizers	2,925
12	Paper, paperboard, and articles	2,825
	Tanker Service	
1	Mineral fuel, oil, bitumen, etc.	421,485
2	Organic chemicals	14,246
3	Inorganic chemicals, rare earth metal compounds, etc.	6,602
4	Animal or vegetable fats, oils, waxes	3,557
5	Sugars and sugar confectionary	1,909
6	Beverages, spirits, and vinegar	1,024
7	Fertilizers	1,022
8	Cereals	942
9	Miscellaneous chemical products	719
10	Plastics and articles thereof	308
11	Salt, sulfur, earth and stone, lime, etc.	164
12	Prepared vegetables, fruit, nuts, etc.	152

Source: U.S. Oceanborne Foreign Trade—Calendar Year 1994, Maritime Administration, U.S. Department of Transportation, Washington, DC, 1996.

$R = 1250$

$AB = \sqrt{\Delta N^2 + \Delta E^2} = 2276.07$

$T = \ldots = 145.18$ m

$\alpha = 13.25$

$\alpha_{AB} = 67.44$

$\alpha_{BC} = 50.19$

$St\ PC = 5 \times 20 + (AB - T)$
$= 109 + 10.89$

$St\ PT$; $L = 289.07$

$St\ PT = 109 \times 20 + 10.89 + L = 123 + 19.96$

$BC = \sqrt{\Delta E^2 + \Delta N^2}$

$T = \ldots$

$BC - T = 571.99$

$Sta.\ C = StPT + BC$

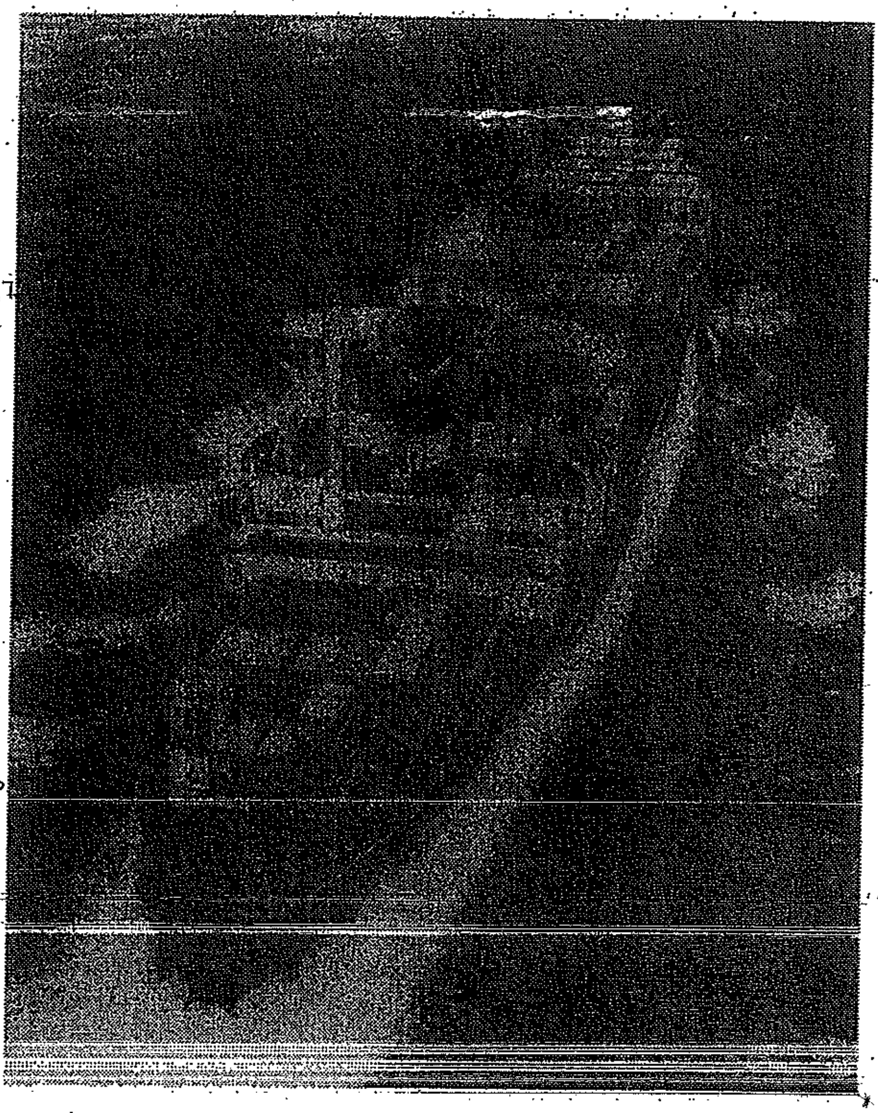


Figure 19-1 A modern container ship. (Courtesy United States Lines.)

   b. Beam, which governs the reach of cargo-handling equipment and influences the width of channels and basins.
   c. Draft, which governs the water depth along berths, in channels, and in basins.
2. Cargo-carrying capacity, which governs the (minimum) storage requirement for full ship loads and influences the handling rate for loading/unloading installations.
3. Cargo-handling gear such as cranes and pumps, which govern cargo-handling rates, in particular for liquid bulk cargo, and which influence the need for port equipment, such as quay cranes and booster pumps. Cranes as well as derricks and winches also restrict the ship motion, which may be tolerated without interruption of cargo-handling operations.
4. Types of cargo units, such as bulk, containers, pallets, and break bulk general cargo, which determine requirements for handling equipment (if ship's gear is not used) and storage.
5. Shape, hull strength, and motion characteristics:
   a. Size and shape of hull and superstructure, which influence berth and fender system layout as well as current and wind forces required for design of moorings and fenders

 $R = 200$ m $C = 1.300$ m

$V = 140$ km/h → 38.89 m/s

b. Ultimate strength of side shell and supporting members, which influences fender design → 75 mm

$\xi \le \frac{E}{G} = \frac{V^2}{gR}$ ⇒ $\xi = 100$ mm (200/b) $\frac{V^2}{gR} = \frac{\xi + EV}{g}$ ⇒ $V_1 = 51.29$ m/s = 185 km/h

c. Ship motions induced by wind waves, swell, or long waves, which influence mooring and fender designs

$V_2 = 19.47$ m/s = 70 km/h (70–185)

d. Superstructure configuration, which influences positioning and design of port-handling equipment to avoid damage to superstructure and handling equipment

6. Mooring equipment, such as ropes, wires, and constant-tension winches, which influences the motion of ships and their mooring forces.
7. Maneuverability at low speed, which influences channel, port entrance, and basin layout as well as the need for harbor tugs.

Typical dimensions and operating characteristics of waterborne vessels are given in Chapter 4.

In the paragraphs that follow, coastal environmental conditions that complicate the engineering design and shorten the useful life of port and harbor facilities will be discussed.

Due to space limitations, it will not be possible to discuss a number of topics that relate to water transportation. Specifically, the important topics of river hydraulics and the design of locks, dams, and canals will not be covered. For information on these topics, the reader may refer to other textbooks or to the publications listed at the end of this chapter.

(1) ASD = 95 m, V = 20 km/h = 27.22 m/s, G = −2%, f = 0.3

DESIGN FOR THE COASTAL ENVIRONMENT

MRSSD = $d_b + d_r$ = 89.87

$d_b = \frac{V^2}{254(f+G)}$, $d_r = Vt$

البيئة الساحلية

19-5. THE COASTAL ENVIRONMENT

ASD > MRSSD

The design of durable port and harbor facilities is one of the most challenging problems that faces the engineer. The environment of the seacoast is harsh and corrosive, and water transportation facilities must be designed to withstand the various destructive biological, physicochemical, and mechanical actions that are inherent to the coastal environment. Enormous forces of winds, waves, and currents are imposed on port and harbor structures. Wood structures must withstand the forces of decay and the attack of termites and other biological life. Concrete structures must be designed and constructed to highest engineering standards to prevent rusting of the reinforcement and spalling of the concrete. Without protection, steel structures corrode and do not last long in the coastal environment.

(3) f, not given & $a = 3.4$ m/s²

⇒ $0.278Vt + \frac{V^2}{254(\frac{a}{9.81} \mp g)}$ ⇒ 137.75, MRSSD Less

19-6. WIND

Wind is the approximate horizontal movement of air masses across the earth's surface. Winds result from changes in the temperature of the atmosphere and the corresponding changes in the air density. Wind exerts a force against objects in its path that depends on the shape and area of the object, the air density, and the square of the wind velocity. The equation for the calculation of this force is

$95 = 0.278V(2.5) + \frac{V^2}{254(\frac{3.4}{9.81} + 0.02)}$ → km/h

$$F = C_D A \frac{\rho v^2}{2} \quad N \tag{19-1}$$

$V = 66.32$ km/h

Reduce V from 80 to 66.32

where

C_D = aerodynamic drag coefficient
A = frontal cross-sectional area, m²
ρ = air density, kg/m³, typically about 1.3 kg/m³
v = wind velocity, m/sec

(2) Reduce: $95 = V(7.5) + \frac{V^2}{...}$

$V = 16.97 = 61.09$ km/h

Considerable judgment is required in computing wind forces on coastal structures and port facilities. Wind forces beyond certain limits make it difficult or impossible to operate cranes, belt conveyors, pneumatic systems, and other loading and unloading equipment. Manufacturers of materials-handling equipment usually provide information on maximum allowable wind speeds. Generally speaking, loading equipment will not be used when winds exceed about 25 km/hr (15 mph), and ships usually will not remain alongside a wharf during a severe storm.

19-7. WAVES

[redacted] Criteria for Vmax

Waves

السبب الرئيسي لتشكل الأمواج هو الرياح ؟

While the design of buildings and other structures must accommodate wind loads, our principal interest in wind in coastal design lies in its contribution to the formation of waves. Waves may be caused by earthquakes, tides, and man-made disturbances such as explosions and moving vessels; however, the waves of principal interest in coastal design are those formed by winds.

When wind moves across a still body of water, it exerts a tangential force on the water surface that results in the formation of small ripples. These irregularities tend to produce changes in the air stream near the water surface. Pressure differentials in the air stream are formed that cause the water surface to undulate. As the wind continues, this process is repeated and the waves grow.

The form and size of water waves have been the subject of considerable scientific observation and research. There is now general acceptance that the surface of deep-water waves is approximately trochoidal in form. According to the trochoidal theory, wave movement can be described by assuming that individual particles are not translated but rotate in a vertical plane about a horizontal axis. This is consistent with the observed tendency of a floating object in deep water to rise and fall and oscillate but not to be translated by the waves.

Figure 19-2 shows the surface of an ideal deep-water wave as a trochoid formed by the rotating particles of water. The trochoid is described by a point on a circle that rotates and rolls in a larger concentric circle. The center of the circle moves along a line that lies above the still water level. The amount this line is elevated above the still water level

1) Deep water حركة دائرية

2) shallow حركة انتقالية

[redacted]

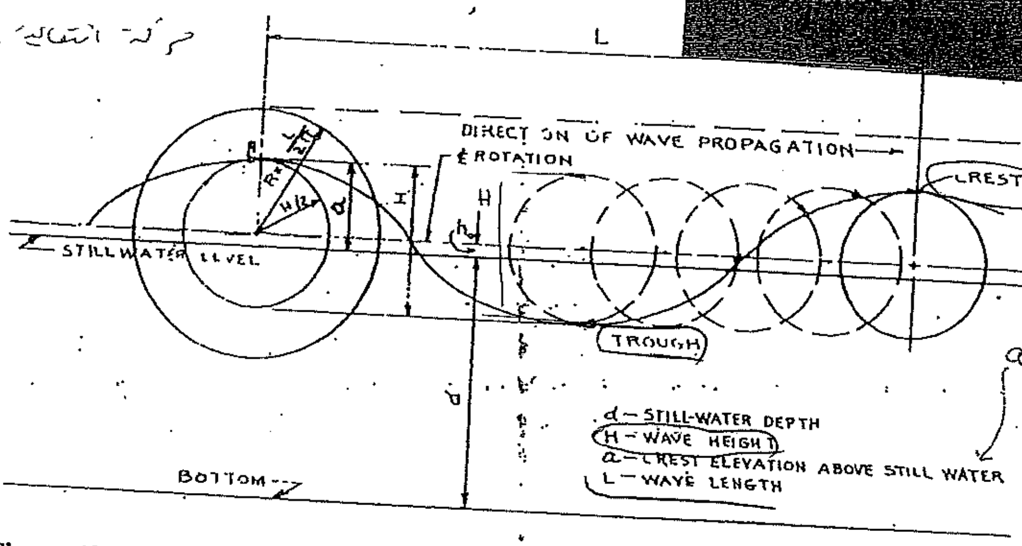


Figure 19-2 Deep-water wave characteristics. (Source: Alonzo Quinn, *Design and Construction of Ports and Marine Structures*, Second Edition, McGraw-Hill, 1972. By permission of the publisher.)

depends on the wave steepness. The diameter of the smaller circle is equal to the wave height, and the circumference of the larger circle equals the wave length. The line thus described represents the wave surface. Particles beneath the wave surface also tend to rotate similarly in circular paths, but the radii decrease rapidly and roughly exponentially with depth...

The speed of propagation of the waveform, or wave celerity, in meters or feet per second, is given as

General

$$\boxed{v = \frac{L}{T}} \tag{19-2}$$

Not for lakes

where

L = wavelength, or the distance between consecutive crests, m (ft)
T = wave period, or the time for a wave to travel one wavelength, sec

The speed of waves in deep water[3] is approximately equal to the velocity acquired by a body falling freely through a height equal to one-half the radius of a circle, the circumference of which equals the wavelength. Thus

Deep Water $d > \frac{L}{2}$

$$\boxed{v = \sqrt{2g\frac{1}{2}\frac{L}{2\pi}} = \sqrt{\frac{gL}{2\pi}}} \tag{19-3}$$

In this equation, g is acceleration due to gravity, which is 9.80 m/sec^2 (32.2 ft/sec^2).

In shallow and transitional water, the paths of water particles are influenced by the frictional forces of the sea bed. This causes the orbit of the water particles to become approximately elliptical with the major axis horizontal.

Waves in shallow and transitional water are complex, and many theories have been proposed to describe them. It is agreed that the velocity of such waves is a function of the water depth.

The following equation should give suitable estimates of wave velocities in depths less than $\frac{1}{25}$ of the wavelength:

shallow $d < L/25$

$$\boxed{v = \sqrt{gd}} \tag{19-4}$$

where

d = water depth, m (ft)

A general equation for wave velocity in shallow and transitional water (applicable where $\frac{1}{25} < d/L < \frac{1}{2}$) has been proposed by G. B. Airy:

shallow

$$\frac{1}{25} < \frac{d}{L} < \frac{1}{2}$$

$$v = \sqrt{\frac{gL}{2\pi}\tanh\frac{2\pi d}{L}} \tag{19-5}$$

It is noted that Eq. 19-5 generalizes to Eq. 19-3 when d becomes greater than L.

Oblique angle

As a comparison of shallow- and deep-water equations indicate, the velocity of the wave decrease as it moves into shallow water. When waves approach the shore at an oblique angle, the portion of the wave nearest the shore slows down with the result that the wave swings around and tends to become parallel to the shore. At the same time, the

[3]Deep-water waves are defined as those occurring where the depth is greater than one-half the wavelength.

Wave Refraction -

wavelengths decrease as the wave period remains constant. This phenomenon is known as wave refraction. The U.S. Navy Hydrographic Office has published a graphical procedure for determining the direction of waves and the lengths of refracted waves [5].

When waves move into shallower depths, as along the coast, the orbits of the particles become distorted due to the friction exerted by the bottom. This causes the major axis of the elliptical path to tilt shoreward from the horizontal, and the wave gradually transforms from a purely oscillatory wave to a wave of translation. (See Fig. 19-3.) It is at this point that waves are capable of exerting great forces against bulkheads, breakwaters, and other coastal structures. Techniques for estimating these forces will be described in Chapter 20.

Wave heights have been found to vary with wind velocity and fetch, the straight-line stretch of open water available for wave growth. It is a region in which wind speeds and direction are reasonably constant. Several empirical equations have been proposed for the estimation of maximum wave height. Two such equations were published by Molitor [6] in 1934 for estimating maximum wave heights in inland lakes:

lackes

$$H_{max} = \begin{cases} 0.17\sqrt{UF} & \text{for } F > 20 \text{ miles} \quad (19\text{-}6) \\ 0.17\sqrt{UF} + 2.5 - \sqrt[4]{F} & \text{for } F < 20 \text{ miles} \quad (19\text{-}7) \end{cases}$$

1 mile = 5280 ft

where

H_{max} = maximum wave height, ft

F = fetch, statute miles (width of storm)

U = wind velocity, statute mph

1 km = 0.62137 miles

INLAND LAKES

These equations were based on similar equations published by Thomas Stevenson in 1864.

Because of the difficulties inherent in measuring maximum wave heights, observers customarily refer instead to the significant wave height. Significant wave height is defined as the average height of the highest one-third of the waves for a stated interval. The maximum wave height, which should be used for design, is equal to approximately 1.87 times the significant height.

Several researchers have developed theoretically based, empirical equations relating significant wave height to fetch and wind velocity. For example, Wilson [7] fitted a curve to deep-water height data from 14 sources and reported the following dimensionless relationship between the significant wave height and fetch and wind velocity:

$H_{max} = 1.87\ H_{significant}$

$$\frac{gH}{U^2} = 0.26 \tanh\left[\frac{1}{10^2}\left(\frac{gF}{U^2}\right)^{1/2}\right] \qquad (19\text{-}8)$$

In Eq. 19-8 g is acceleration due to gravity.

The Corps of Engineers [8] has published the forecasting curves shown as Figs. 19-4 and 19-5 for predicting wave heights and wave periods, respectively. These idealized, dimensionless plots for wave growth include adjustments for shallow water to account for

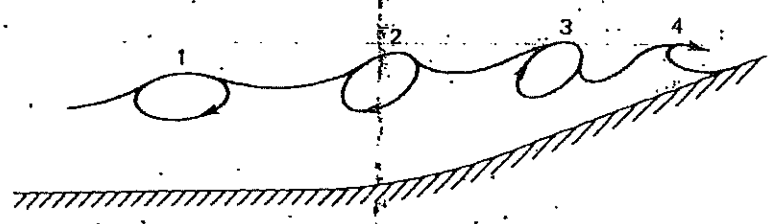
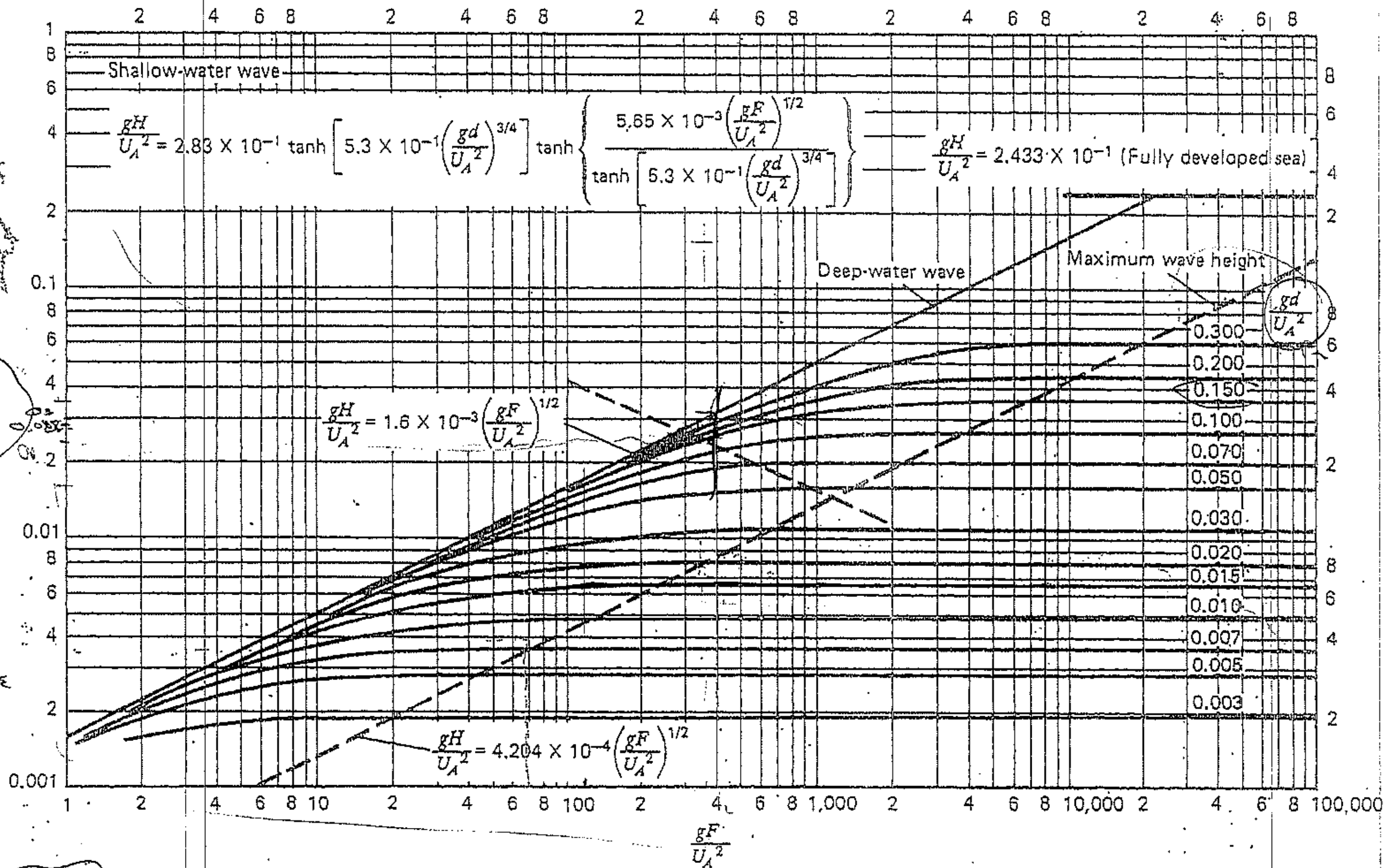


Figure 19-3 Development of a wave of translation.

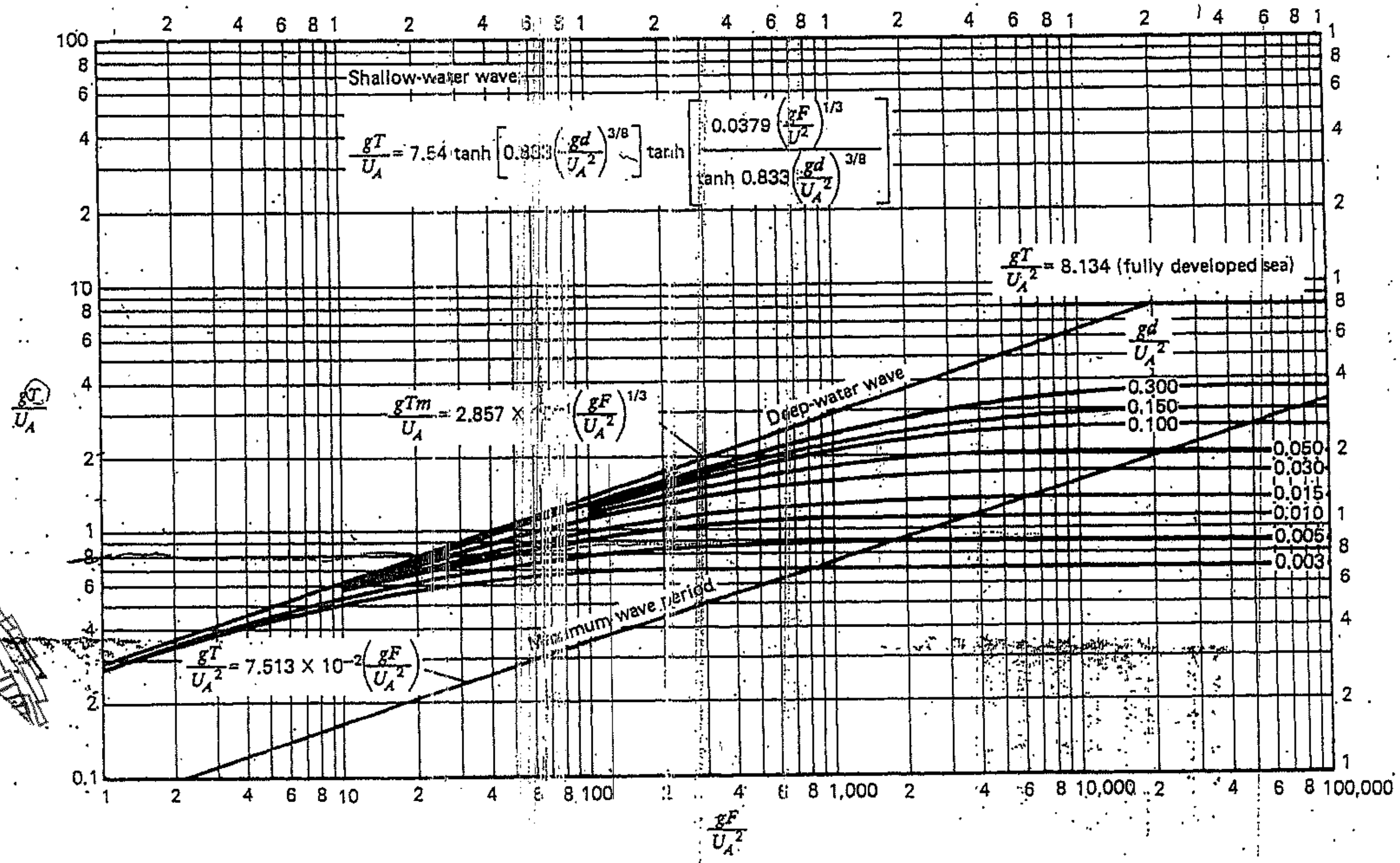
Significant Height



١) تحديد الخط المائل
٢) تحديد ال X-axis
٣) يطلع ضو نكون في x حد الخط المائل

deep water & Shallow and water.

Figure 19-4 Forecasting curves for wave height. Constant water depth. (Source: Reference 8.)



deep water & shallow.

Figure 19-5 Forecasting curves for wave period. Constant water depth. (Source: Reference 8.)

energy lost due to bottom friction and percolation. The curves are expressed in terms of the fetch, the water depth, and a function of the wind speed called the *wind stress factor.*

Assumption: 10 m elev. measurements

Below an elevation of about 1000 m, the frictional effects due to the presence of the ocean distort the wind field. In that zone, wind speed and direction depend on the elevation above the mean surface, air–sea temperature differences, and other factors. For example, it is usually assumed that the winds are measured at the 10-m elevation. If this is not the case, the wind speed must be adjusted accordingly.

The growth of wind-generated waves is most directly explained by the surface stress, which depends on the wind speed but may also be affected by a number of other factors. The wind effects on wave growth in the wave forecasting curves are therefore expressed in terms of U_A, the wind stress factor, which accounts for the nonlinear relationship between wind speed and wind stress. The Corps of Engineers [8] suggests that the following equations be used to convert the adjusted wind speed to the wind stress factor U_A:

wind velocity

$$U_A = 0.71U^{1.23} \quad (U \text{ in m/s}) \tag{19-9a}$$

$$U_A = 0.589U^{1.23} \quad (U \text{ in mph}) \tag{19-9b}$$

NOTE

The plots shown in Figures 19-4 and 19-5 were developed by defining the wave height parameters in terms of the standard deviation of the sea surface *elevations* rather than the heights of individual waves in a record. In this approach, the variance of the sea surface elevations is proportional to the average energy of a spectrum of waves, and the significant wave height is equal to four times the standard deviation. These parameters represent a fundamentally different class called "energy-based" or "spectrally based" parameters. Experimental results have shown, however, that in deep-water conditions the spectrally based significant wave height is practically equal to the significant wave height as determined from the heights of individual waves [8].

The nomogram reproduced as Fig. 19-6 shows wave prediction curves of empirical values that can be used to check the reasonableness of the theoretical solutions of Fig. 19-4 and 19-5. The nomogram can be used to estimate the significant height of deep-water waves. These curves provide a direct method for estimating wave heights in simple wind fields in deep-water areas away from the coasts. However, near the coast, the significant wave methods may produce overestimates of wave heights due to failure to consider the directional spreading of the waves [9].

In shallow or transitional water, the water depth affects wave generation. Where waves are generated in such areas, smaller wave heights and shorter wave periods will be experienced. Several researchers [10–12] have contributed to the development of forecasting methods for waves generated in shallow waters. The Corps of Engineers [8] has published a group of graphs for wave forecasting in relatively shallow waters. The graphs, exemplified by Fig. 19-7, were developed by successive approximations in which wave energy is added due to wind stress and subtracted due to bottom friction and percolation.

Consider the following examples, which illustrate the use of the wave forecasting graphs.

EXAMPLE 19-1

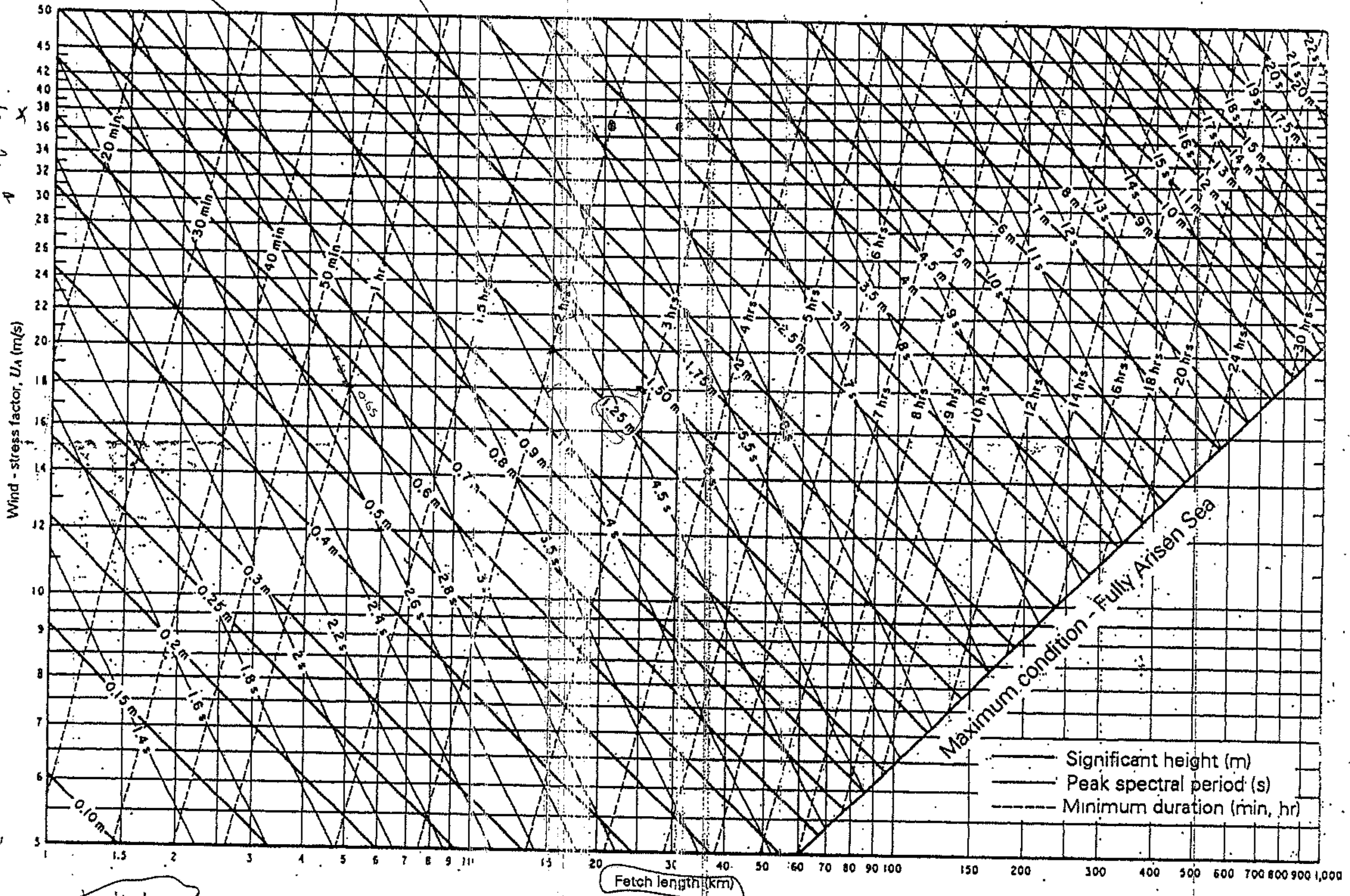
F U_A d

Given a fetch of 15 km, a wind stress factor of 20 m/sec, and a mean water depth of 6 m, determine the significant wave height and period:

$$\frac{gd}{U_A^2} = \frac{9.8(6)}{(20)^2} = 0.147$$

ما اقرب خط غامق H
اقرب خط قاطع T

U_A in m/s



check deep

Figure 19-6 Nomogram of deep-water significant wave prediction curves as functions of wind-stress factor, fetch length, and wind duration. (Source: Reference 8.)

Deep water

U_A in m/s

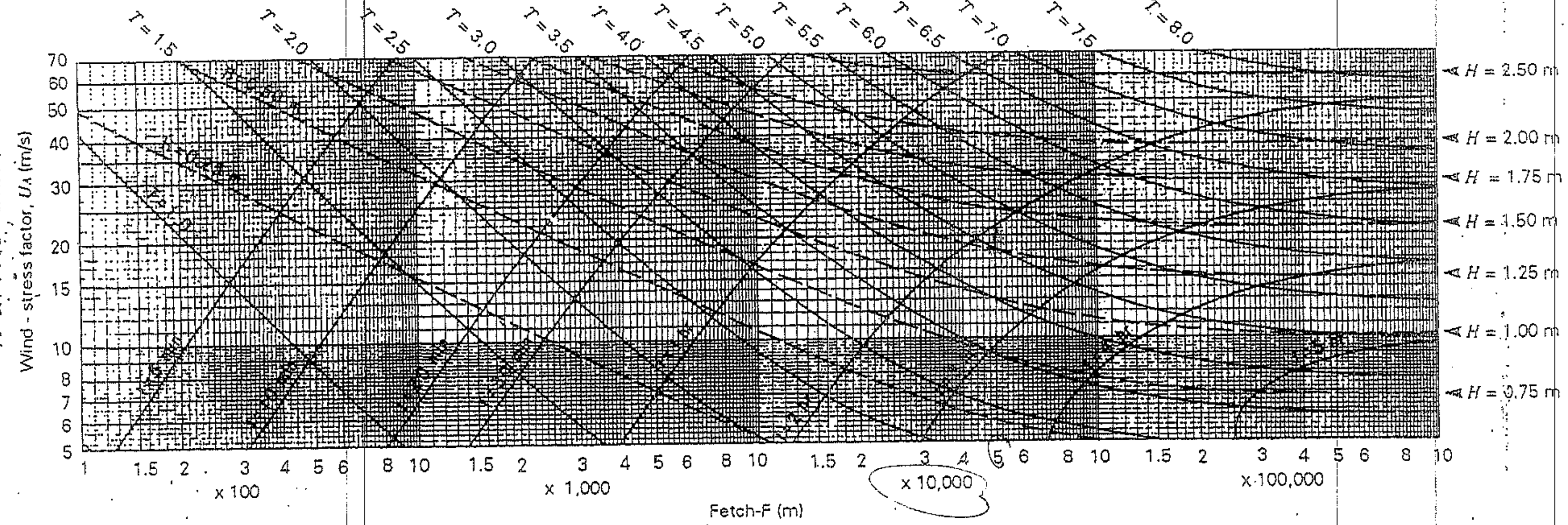


Figure 19-7 Forecasting curves for shallow-water waves with constant depth = 6.0 m. (Source: Reference 8.)

Shallow Water
w/ d=6m

$$\frac{gF}{U_A^2} = \frac{9.8(15{,}000)}{(20)^2} = 368$$

From Figs. 19-4 and 19-5

$$\frac{gH}{U_A^2} = 0.025 \quad \text{and} \quad \frac{gT}{U_A} = 1.8$$

$$H = \frac{(20)^2(0.025)}{9.8} = 1.02 \text{ m}$$

$$T = \frac{(20)(1.8)}{9.8} = 3.7 \text{ sec}$$

Checking the results of Example 19-1 with Figure 19-7 yields

$$H = 1.0 \text{ m} \quad \text{and} \quad T = 3.6 \text{ sec}$$

EXAMPLE 19-2

deep

F U_A

Given a fetch of 25 km, a wind stress factor of 18 m/sec, and a mean water depth of 10 m, determine the significant wave height and period. If the storm that produced the winds lasted only 1.0 hr, how would this affect the answer?

$$\frac{gd}{U_A^2} = \frac{9.8(10)}{(18)^2} = 0.30$$

$$\frac{gF}{U_A^2} = \frac{9.8(25{,}000)}{(18)^2} = 756$$

From Figs. 19-4 and 19-5

$$\frac{gH}{U_A^2} = 0.037 \quad \text{and} \quad \frac{gT}{U_A} = \cancel{3.3}\ 2,5$$

$$H = \frac{(18)^2(0.037)}{9.8} = 1.22 \text{ m}$$

$$T = \frac{(18)(3.3)}{9.8} = \overset{4.2}{\cancel{6.1}} \text{ sec}$$

If the storm lasted only 1.0 hr, Fig. 19-6 indicates that $H = 0.65$ m and $T = 2.8$ sec. This is known as a duration-limited wave.

2km
4 mile/h

19-8. CURRENTS

Much of water movement, as in waves, occurs in form rather than in the translation of water particles. Since water is viscous, however, the rotating particles that constitute waves do not return to their original position but rather drift in the direction of the wave movement. This flow, which occurs at a velocity smaller than the velocity of the wave itself, is called a current.

Offshore currents may result from hydraulic head when water is piled up along the coast because of mass translation associated with tides or waves. An example of this type of current is the Gulf Stream, which flows northeasterly at a speed of about 6 km/hr (4 mph) between the southern tip of the Florida peninsula and the Bahama Islands.

The engineer's principal interest in currents lies in his or her efforts to stabilize erodible shoreline and to maintain navigable inlets.

354 356

19-9. TIDE

Some knowledge is required by the designer of port and harbor facilities of the nature and effects of tides. The tide is the alternate rising and falling of the surface of the oceans, gulfs, bays, and coastal rivers caused by the gravitational attraction of the moon and sun. In most places, such as on the Atlantic coast of the United States, the tide ebbs and flows twice in each lunar day (24 hr and 50 min). Larger than usual tides, called spring tides, occur when the sun and moon act in combination as when there is a new moon or full moon. Smaller than usual tides, called neap tides, occur when the moon is at first or third quarter.

In addition to the effects of the moon and sun, the magnitude and nature of a tide at a given location and time will be influenced by:

1. Geographical location
2. Physical character of the coastlines سواحل
3. Atmospheric pressure
4. Currents

open sea

At certain inland and landlocked seas, such as the Mediterranean Sea and the Gulf of Mexico, the tides are practically negligible. At other places, such as the Bay of Fundy, local topographical peculiarities contribute to tides as high as 30 m (100 ft).

Tidal charts and tables are published for the various ports of the world by the National Geodetic Survey, the British Admiralty, and other organizations.

19-10. PHYSICAL AND MATHEMATICAL MODELS

There are many problems in port planning and design that do not lend themselves to simple and straightforward analysis. Such problems are often addressed with physical or hydraulic models. Physical models may be used to facilitate the overall design of a harbor to determine the best locations and dimensions of structures to provide required protection against wave attack. Such models allow port planners to determine the proper types and alignments of coastal structures; the locations, shapes, and widths of navigation openings; and the effects of proposed dredging operations within the harbor. They are also used to evaluate the structural design of coastal facilities and to study inlet stability and hydraulics.

Figure 19-8 shows a physical model of Murrells Inlet in South Carolina. A spectral wave generator produces waves that simulate the actual waves that approach the inlet. The model, which was developed by the U.S. Army Engineer Waterways Experiment Station, has a vertical scale of 1 : 60 and a horizontal scale of 1 : 200.

An important recent development in port and harbor planning has been the development of mathematical modeling by means of advanced digital computers. Mathematical models can take into account virtually any physical phenomena that can be described in physical form. Thus, with mathematical models, it is possible to describe many phenomena that would be impossible or extremely costly to describe with a physical model. For

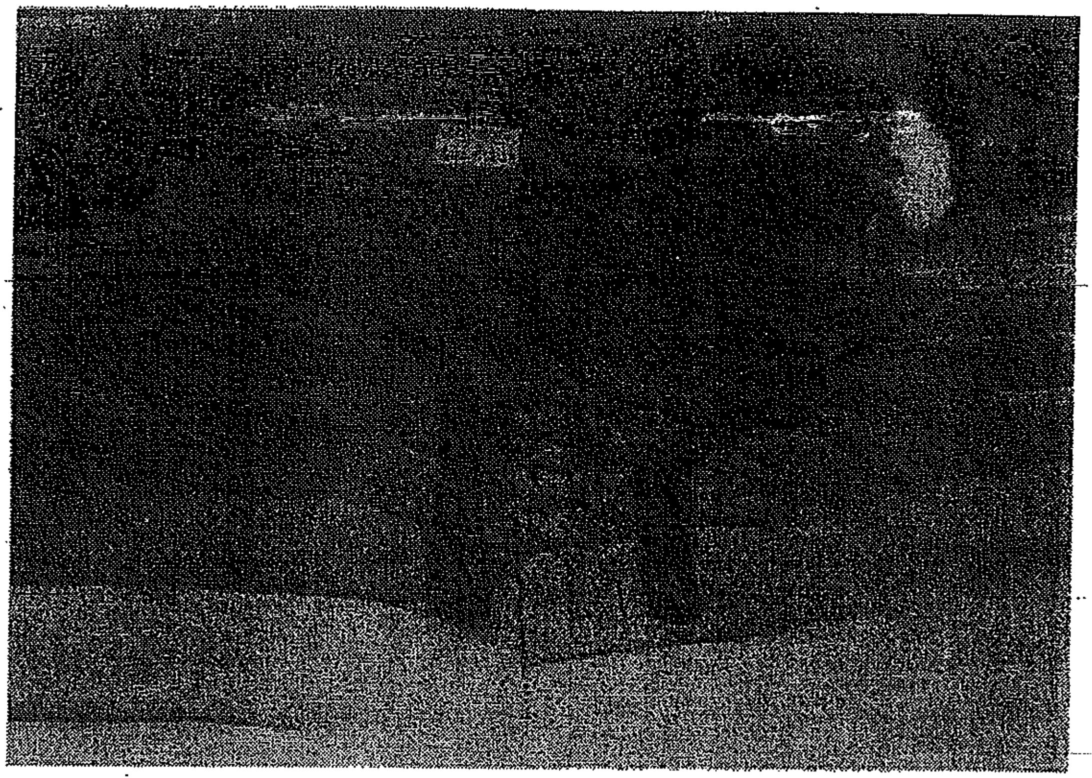


Figure 19-8 A physical model of Murrells Inlet in South Carolina. (Courtesy U. S. Army Corps of Engineers.)

example, mathematical models can describe the effects of winds on currents and water levels in shallow waters and on ships maneuvering at slow speeds [3]. Mathematical models are also less costly to maintain and to remobilize and are not subject to the errors of "scale effect" that may be present in physical models. Nevertheless, at least for the foreseeable future, mathematical models are likely to complement rather than replace physical models in providing solutions to coastal hydraulic problems.

Figure 19-9 shows a computer-plotted wave pattern for the port of Valencia, Spain.

DETERIORATION AND TREATMENT OF MARINE STRUCTURES

Port and harbor structures must withstand some of the most destructive environmental conditions found in the world. This is especially true of piles and other elements of substructures that are subject to attack by marine life as well as the corrosive effects of salt and sea. Generally these destructive effects are most pronounced in a seawater environment, and some of the destructive agents, such as certain species of marine borers, are not found in freshwaters.

In Sections 19-11, 19-12, and 19-13 the causes of deterioration and methods of protection of wood, concrete, and steel waterfront structures will be discussed. Reference 13 gives a more extensive treatment of this important subject.

19-11. DETERIORATION AND TREATMENT OF WOOD STRUCTURES

Timber used in coastal structures may be damaged by three forms of attack: decay, insects (termites and wharf borers), and marine borers. The decay and insects tend to damage structures above the water level, while marine borers attach below the water level.

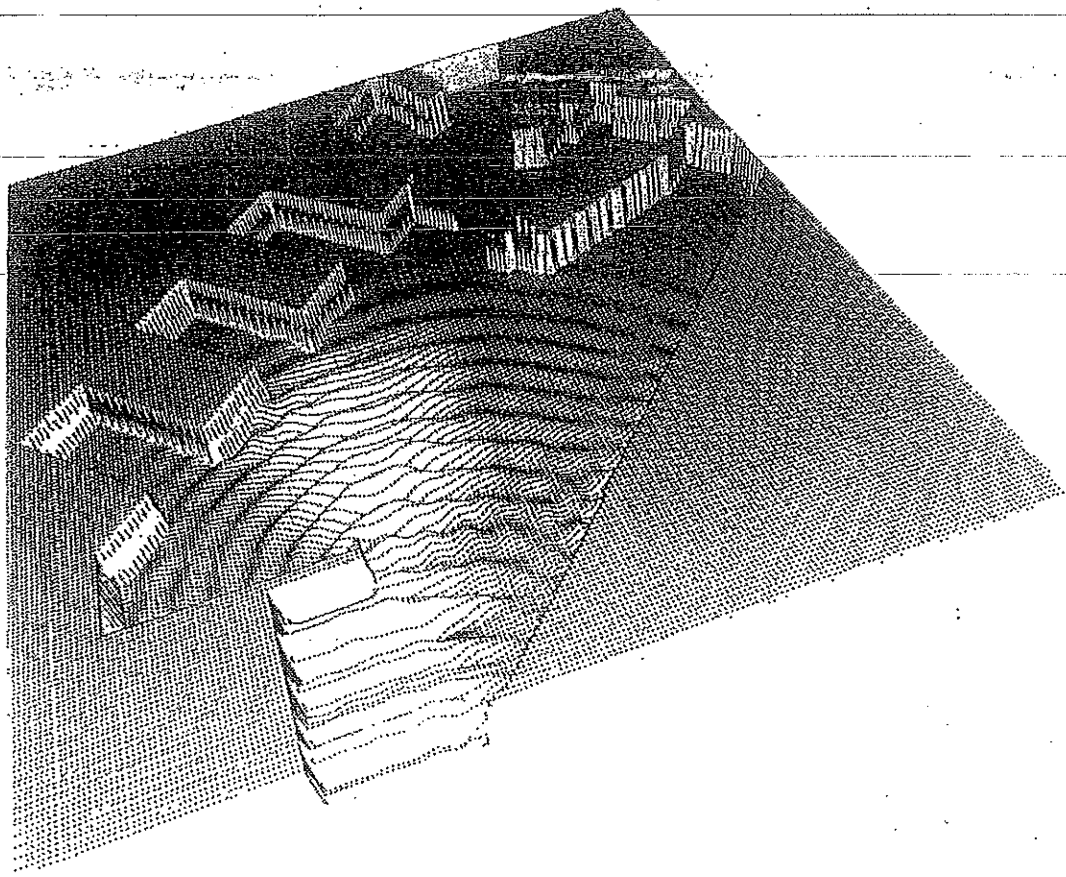


Figure 19-9 A computer-plotted wave pattern for the port of Valencia, Spain. (Source: Reference 4.)

Decay. All forms of decay are caused by certain low forms of plant life called fungi. The fungi consume certain substances in the wood, causing it to disintegrate. The species of fungi that cause decay require food, air, adequate moisture, and a favorable temperature. Poisoning the food supply by impregnating the timber with a suitable preservative is the most common method of preventing decay.

Termites. Sometimes called "white ants," termites are not really ants, but resemble them somewhat in appearance and method of life. There are two types of termites: (1) the "subterranean" type that requires moisture and therefore must have access to the ground at all times and (2) the "dry wood" type that flies and does not need contact with the ground. The subterranean termites are widely distributed throughout the world, but the dry wood termites tend to be found only in the warmer climates.

The chief food of termites is cellulose, which they obtain from dead wood. Most species of timber are subject to the attack of termites. It is therefore necessary, to prevent termite damage, to use commercial timbers that have been properly treated; or to insulate all wood from the ground so that the termites cannot attack it. However, insulation will not prevent attack by dry wood termites.

Wharf Borers. These animals can do extensive damage to wood structures but are not of as much concern as other organisms that attack timber. The damage is caused by the young of the winged beetle. The beetle lays its eggs in the cracks and crevices of the timber, and the larvae or worms hatched from these eggs are the cause of the damage to the timber. Wharf borers do not work below water level but more commonly in timber that is not far above high water or in timber that is wet by salt spray at times.

Marine Borers. There are two main divisions of these very destructive animals: (1) the molluscan group, which is related to the oyster and clam, and (2) the crustacean group, which is related to the lobster and crab. Their methods of attack on timber are completely different. The molluscan group enters the timber through a tiny hole, and the animals grow as they destroy the interior of the timber. The crustacean group destroys the timber from the outside. The attack of the molluscan borers can only be detected by the most careful inspection of the surface or by cutting into it. The attack of the crustacean borers is easily seen and measured by surface inspection, and the rate of destruction in heavy attacks is less rapid by the crustaceans than by the molluscans.

Molluscan borers are classified biologically into several genera and many species. One group, the toredo or shipworm, has grayish, slimy, wormlike bodies with the shells on the head used for boring. The size of the mature animals of the common species may be as large as 25 mm (1 in.) in diameter and 1.2 to 1.5 m (4 to 5 ft) long, but more commonly they are about 9 mm (⅜ in.) in diameter and 125 to 150 mm (5 to 6 in.) in length. In areas of heavy attack, these borers may totally destroy an unprotected pile's bearing value in 6 to 8 months.

Another group, the pholad, also uses its shells for boring, but the body of the animal is enclosed by the shells. Some species of the group bore in soft rock and mud and even concrete. The entrance holes made by this group tend to be larger than those made by the toredo group but may still be hard to find by surface inspection. This group is most commonly active in tropical and semitropical waters.

Crustacean borers are classified into several genera and species. One of the species resembles a wood louse; another is related to the ordinary sand hopper. These borers have a body that is typically 3 to 13 mm (⅛ to ½ in.) in length and a width of about one-third to one-half of the length. Crustacean borers exist in a wide variety of climates and environments and may be present in either saltwater or freshwater. Hundreds of these animals per square inch have been counted on timber under heavy attack. They commonly destroy timber, gnawing interlacing branching burrows on the surface of the timber.

Many methods have been tried to protect timber from marine borer attacks, including covering the timber with metal sheathings or encasing it in concrete, cast iron, or vitrified pipe. The most common and reliable form of protection has been impregnation of the timber with a toxin such as coal tar creosote. This type of treatment also protects the wood from decay, termites, and wharf borers.

19-12. DETERIORATION AND PROTECTION OF CONCRETE STRUCTURES

Concrete piles that are entirely embedded in earth generally are not subject to significant deterioration. However, in isolated instances, concrete piles may be damaged by the percolation of groundwater that contains destructive chemicals from industrial wastes, leaky sewers, or leaching from storage piles of coal or cinders containing acids or other destructive compounds.

Reinforced concrete piles above the ground surface may be damaged by weathering and destructive elements carried in the air. Moisture in the air may penetrate permeable concrete and reach the reinforcement, causing it to rust and spall the sides and edges of the pile. In addition, damage to reinforced piles in waterfront structures may be caused by:

1. Chemical action of polluted waters
2. Attack by molluscan borers
3. Abration by floating objects or scouring sands
4. Frost action

Because of the harshness of the coastal environment, special care is required to ensure that the concrete meets high standards of materials and workmanship. Experience has shown that the composition of the Portland cement, the quality of the aggregates, the amount of cover over the steel, and the workmanship in mixing and placing the concrete are the most important factors that affect the durability of the concrete piles.

19.13 DETERIORATION AND PROTECTION OF STEEL STRUCTURES

Steel piles entirely embedded in relatively impervious earth receive little damage from corrosion. From a level of about 0.6 m (2 ft) below the ground surface downward, atmospheric oxygen is blanketed off by the surrounding soil, inhibiting progressive corrosion. Occasionally, however, steel piles that are completely embedded may deteriorate if the surrounding earth or groundwater contains corrosive compounds.

Steel piles protruding from the ground into open water are subjected to severe deterioration in a saltwater environment. Steel piles in freshwater generally do not require protection.

Steel piles in seawater tend to experience less corrosion if in protected waters than if subjected to wave action in the open ocean. Deterioration may be worse because of the abrasive action of waterborne sand agitated by waves and currents. This condition usually exists only in shallow waters where wave and tidal action is most prevalent. Destructive organic substances consisting of decayed marine life deposited on the bottom may also cause severe deterioration in a narrow zone at the mud line. Where either of these conditions exist, it may be desirable to protect the piling by some form of encasement in the vicinity of the mud line.

Above the mud line, corrosion is usually more active near the water surface where the oxygen content of the water is greatest. It tends to be more severe in regions exposed to wetting and drying or to saltwater spray. In these areas, various coatings and paints or, in extreme cases, concrete casings are used to inhibit corrosion.

PROBLEMS

1. Estimate the velocity of a 45-m-long wave in water 25 m deep.
2. Estimate the velocity of a 120-ft-long wave in water 65 ft deep.
3. Estimate the velocity of a 60-m-long wave in water 2.5 m deep.
4. Estimate the velocity of a 200-ft-long wave in water 7.0 ft deep.
5. Estimate the velocity of a 50-m-long wave in water 4.0 m deep.
6. Estimate the velocity of a 155-ft-long wave in water 12 ft deep.
7. Estimate the significant wave height for a fetch of 25 km and a wind velocity 12 m/sec.
8. Estimate the maximum deep-water wave height for a fetch of 170 statute miles and a wind velocity of 18 statute mph.
9. Given a fetch of 36 km, a wind speed of 72 km/hr, and a mean water depth of 15 m, determine the significant wave height and period. If the storm that produced the winds lasted only 1.5 hr, how would this affect the answer?
10. Given a fetch length of 24 km, a wind speed of 58 km/hr, and a constant water depth of 6 m, determine the significant wave height and wave period. Assume shallow water.

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Chapter 20

Planning and Design of Harbors

20-1. INTRODUCTION

A harbor is a partially enclosed area of water that serves as a place of refuge for ships. The term *port* refers to a portion of a harbor that serves as a base for commercial activities. Harbor and coastal structures are the means by which protection from waves and winds is provided and the erosion of beaches and coastlines is controlled. Port facilities make it possible for ships to obtain fuel and supplies, to be repaired, and to transfer passengers and cargo.

This chapter will deal with the planning and design of coastal and harbor structures; Chapter 21 will be concerned with the planning and design of port facilities.

20-2. CLASSES OF HARBORS

Harbors may be classified into one of several categories according to function and protective features. There are many natural harbors in the world where protection from storms is provided by natural topographical features. Natural harbors may be found in bays, inlets, and estuaries and may be shielded by offshore islands, peninsulas, or reefs. Other natural harbors are protected by virtue of the fact that they are located in river channels some distance from the sea. Several famous world harbors are located as much as 80 km (50 miles) inland.

When sufficient protection from storms has not been provided by nature, it may be provided by the construction of breakwaters and jetties. Harbors thus formed are known as artificial harbors. Planning considerations and design criteria for breakwaters and jetties will be discussed in Section 20-4.

Some degree of protection from storms is provided by harbors of refuge and roadsteads. These areas are easily accessible but generally offer less protection than does a harbor. The term *harbor of refuge* refers to convenient protected anchorage areas that are usually found along established sea routes and dangerous coasts. A *roadstead* is a tract of water that is protected from heavy seas by a bank, shoal, or breakwater. Port facilities are not provided in harbors of refuge or roadsteads, although these facilities may be provided within a nearby harbor.

Additional classifications of harbors include commercial harbors, which provide protection for ports engaged in foreign or coastwise trade, and military harbors, within which the dominant activity is the accommodation of naval vessels.

20-3. DESIRABLE FEATURES OF A HARBOR SITE

There are at least five desirable features of a harbor site:

1. Sufficient depth
2. Secure anchorage
3. Adequate anchorage area
4. Narrow channel entrance in relation to harbor size
5. Protection against wave action

The depth of harbor and approach channel should be sufficient to permit fully loaded ships to navigate safely at the lowest water. Obviously, the harbor depth required depends principally on the draft[1] of the ships using the harbor.

The harbor and channel depth below the lowest low water should be at least the maximum draft anticipated plus an additional 1.5 m (5 ft), approximately. This additional depth is to allow for the tendency of a ship to "surge" when in motion and to provide a clearance of at least 1 m (3 ft) below the ship's keel as a factor of safety.

A summary of the average drafts of various types of ships in the world's ocean fleet is given in Chapter 4, along with average drafts of typical inland waterway vessels. Generally, the average draft of oceangoing ships varies from 6.7 to 11.1 m (21 to 37 ft) depending on ship type, while the draft of the largest commercial oceangoing ship is 28.6 m (94 ft) [1]. Few if any ocean ports have channel depths to accommodate the largest tankers, and special provisions must be made to unload these vessels offshore to smaller tankers or by means of pipelines.

Generally, ocean ports maintain harbor and channel depths of 11 to 12 m (35 to 40 ft); however, the trend to larger ships indicates that modern ocean harbors may require depths in excess of 12 m (40 ft).

On the inland waterway system, a 2.7-m (9-ft) operating depth is considered standard. Channel depths on the inland system vary a great deal, however, and about 26 percent of the inland waterway channel miles has a depth of less than 1.8 m (6 ft) while approximately 18 percent has a depth of 4.3 m (14 ft) and over [2].

Harbor site & Soil conditions

Conceivably, the selection of a harbor site may be influenced by the soil conditions along the bottom of the anchorage area. Generally, firm cohesive materials provide good anchorage, while light sandy bottoms are poor anchorage areas. Other factors being equal, more anchorage area will be required when poor bottom soil conditions prevail. This follows from the fact that maximum resistance to ship movement occurs when the anchor cable is as nearly horizontal as possible.

The shape and extent of the anchorage area is dependent principally on five factors:

1. ~~The maximum number of ships to be served~~
2. The size of the ships
3. The method of mooring
4. Maneuverability requirements
5. Topographic conditions at the proposed site

Because objectionable waves will be generated in large harbors, artificial harbors should be built as small as possible and still be consistent with the needs for convenient safe maneuvering and mooring.

Space requirements for ships vary a great deal depending on the method of mooring. A ship that is secured by a single anchor will occupy a circular area with a radius of the ship

[1]Draft is the vertical distance between the waterline and the keel. Unless otherwise noted, when the word draft is used here, it will refer to the full load draft.

Vehical owner per 1,000
USA 786, Euro 500-600, Isr: 330
Jordan: 177, Palestine: 35, Bangladesh: 3

length plus approximately three times the water depth. Thus, a 183-m- (600-ft-) long ship anchored by a single anchor in a harbor 15 m (50 ft) deep will require about 16 hectares (40 acres) of anchorage area. Considerably less area is required for a ship that is secured by two anchors. Such a ship will occupy a circular area with a diameter little more than the ship length, or about 2.6 hectares (6.5 acres) for a 183-m (600-ft) ship. However, when a clear area is maintained for the anchor cable, a rectangular area about 300 to 370 m (1000 to 1200 ft) long and about 150 to 185 m (500 to 600 ft) wide is required for a typical merchant freighter. Thus, a merchant freighter secured by two anchors requires a total area of 6.5 hectares (16 acres) of harbor space.

Harbor dimensions per ship

The minimum turning radius for a ship is equal to about the ship length. Thus a typical oceangoing freighter will require an additional 12 to 14 hectares (30 to 35 acres) for maneuvering if a full-size turning basin is provided.

Minimum turning radius of ship

As one might expect, harbors of the world vary a great deal in area, the smallest fishing harbors being less than 4 hectares (10 acres) and the largest harbors being more than 26 km^2 (10 mi^2).

To minimize wave action within the harbor, the harbor entrance should be as narrow as possible provided it meets the requirements for safe and expeditious navigation and provided it does not cause excessive tidal currents. Currents in excess of about 1.5 m/sec (5 ft/sec) will affect navigation adversely and may cause scour of breakwaters and other protective works.

Minimize Waves action

The required entrance width naturally will be influenced by the size of the harbor and the ships that use it. As a rule of thumb, the width of the entrance should be roughly equal to the length of the largest ship using it. While this guideline should be helpful for planning purposes, the entrance width and location used for design purposes should be determined by model tests.[2]

USA => 240 M / Japan => 74 M
# of Veich: China => 78 M / Brazil => 64 M
West bank => 144,000

PROTECTIVE COASTAL WORKS

Produce Ve: China => 12M, USA => 10M, Ger => 5.5M

The term *protective coastal works* will be used here in the broad sense to include:

1. Offshore structures (breakwaters) to lessen wave heights and velocities.
2. Structures that are built at an angle to the shore, such as jetties and groins to control littoral drift.
3. Structures built at or near the shoreline to protect the shore from the erosive forces of waves. In this category are seawalls, bulkheads, and revetments.
4. Natural coastal features such as protective beaches and sand dunes that help to control and dissipate waves without creating adverse environmental effects.

20-4. BREAKWATERS AND JETTIES

Breakwaters are massive structures built generally parallel to the shoreline to protect a shore area or to develop an artificial harbor. A jetty is a structure built roughly perpendicular to the shore extending some distance seaward for the purpose of maintaining an entrance channel and protecting it from waves and excessive or otherwise undesirable currents. Jetties usually are built in pairs, one on each side of a channel or the mouth of a river. Structurally, breakwaters and jetties are similar; however, the design standards for jetties may be slightly lower than are those for breakwaters. This results from the fact that jetties are not subject to direct wave attack to as great an extent as are breakwaters.

[2]Reference 3 provides an excellent case study illustrating the usefulness of hydraulic modeling to quantitatively investigate the effects of future harbor development plans.

death - billion passenger-km
Air Taxi: 0.05, Bus: 0.4, Rail: 0.6, Van: 1.2, Water: 2.6
car: 3.1
space shuttle: 5
bicycle: 44.
foot: 54.2
motor.: 108.9

To avoid redundance, the remaining discussion of this section will focus principally on ~~the planning and design of breakwaters, and little else will be said about jetties. However,~~ one especially important feature relating to the design of jetties should be mentioned, that of hydraulic design. Normally, when longshore transport occurs predominantly in one direction, jetties can cause accretion of the updrift shore and erosion of the downdrift shore. The stability of the shore in the vicinity of inlets can be improved by a process known as *sand bypassing,* which involves artificially nourishing sand-deprived areas by means of land-based dredging plants, floating dredges, or land-based vehicles [4].

Because of the possibility of erosion and accretion due to changes to the velocity and direction of channel currents, the determination of distance between jetties must be made carefully. In making this decision, the designer should study the magnitude and direction of existing tidal currents and the effect that the construction of jetties might have on these currents. Of course, important inputs to this decision will be existing topographical features of the area and the channel width needed for navigation. Unfortunately, few analytical tools are available to help the designer with this problem, and he or she must rely on engineering judgment and, whenever feasible, model studies.

While a wide variety of breakwaters have been built, those successfully employed generally fall into two classes: mound breakwaters and wall breakwaters.

By far the most popular type of breakwater is the rubble mound breakwater. As the name suggests, this type of breakwater consists of a mound of large stones extending in a line from the shore or lying parallel to and some distance from the shore. Typically, this type of breakwater is constructed of stones ranging in weight from 500 lb to more than 16 tons each. The smaller stones are used to construct the core, while the largest sizes, being most resistant to displacement, serve as armor stones that comprise the outer layer of the mound. Commonly, the largest armor stones are used on the seaward side of the breakwater.

Where adequate quantities of armor stone are not available in suitable sizes, precast concrete armor units may be used. Various shapes of these units have been used, including tetrapods, quadripods, and tribars. See Fig. 20-1 and Table 20-1. Some of these shapes are patented and a royalty charge must be paid for their use. A rubble mound breakwater utilizing an armor layer of tetrapods is shown as Fig. 20-2.

The dolos armor unit, illustrated in Fig. 20-3, was developed by E. M. Merrifield, Republic of South Africa, in 1963. This concrete shape is similar to a ship anchor or the letter "H" with one vertical perpendicular to the other [4].

In certain instances, a relatively impervious material is used in the core of the breakwater. When a sand-clay or shale is used for the base and core material, the breakwater is classed as a solid fill structure. A fine-grained material in the core may be used because of economy or in order to prevent the effect of waves passing through the structure. The voids of the upper portion of the core of the breakwater may also be filled with the hot asphaltic concrete or Portland cement concrete in order to improve the stability of imperviousness of the structure.

Relative cost $/ton/km
ocean tanker: .5
Pipeline: 1.0
barges: 1-3
Rail: 4-8
Truck: 6-8
Air: 10-128

Vertical-wall breakwaters constitute a second major class of breakwaters. This class of breakwater differs in concept and design from rubble mound breakwaters. The designer of a vertical-wall breakwater must be concerned with the ability of the total structure to remain stable under the attack of waves, whereas in the case of rubble mound breakwaters, attention must be focused on the stability of the individual stones.

The principal advantage of vertical-wall breakwaters is that less rock is required than is needed for rubble mound construction. These breakwaters, being less massive, provide more usable harbor area and make it possible to have a narrower harbor entrance. On the other hand, rubble mound breakwaters can be constructed on foundations that would be unsuitable for the support of a vertical-wall breakwater. Waves tend to break and be dissipated on the slopes of rubble mound breakwaters, and consequently these

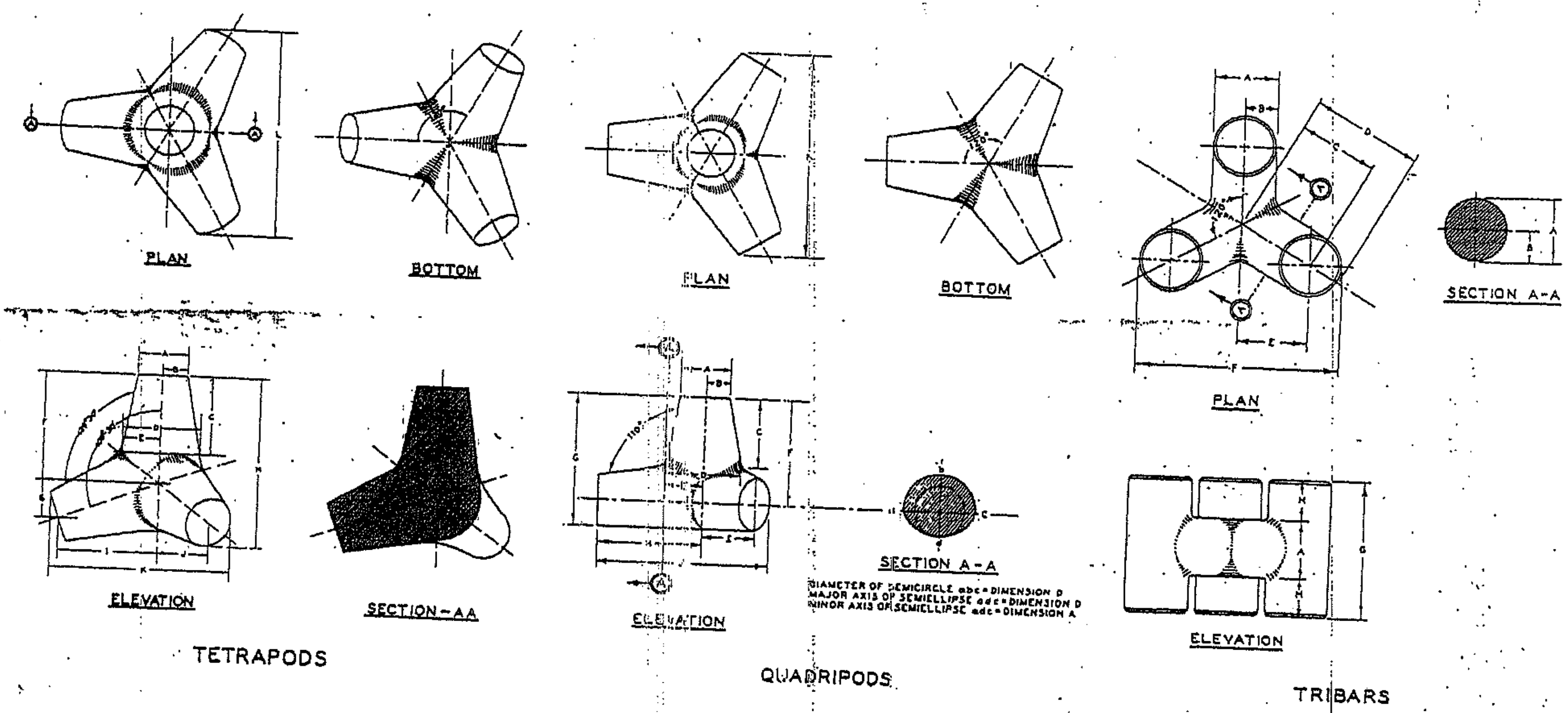


Figure 20-1 Typical concrete armor units. (Source: *Shore Protection Manual*, U.S. Army Corps of Engineers, 1984.)

Table 20-1 Design Information for Concrete Armor Units

Parameter	Tetrapods Small	Tetrapods Medium	Tetrapods Large	Quadripods Small	Quadripods Medium	Quadripods Large	Tribars Small	Tribars Medium	Tribars Large
Volume of armor unit, ft^3	7.14	214.29	571.43	7.14	214.29	571.43	7.14	214.29	571.43
Weight of armor unit, tons[a]	0.53	16.02	42.71	0.53	16.02	42.71	0.53	16.02	42.71
Average thickness of two layers placed pell-mell ft	4.01	12.45	17.26	3.66	11.37	15.77	3.85	11.97	16.60
Number of armor units/1000 ft^2 (2 layers, pell-mell)	280.18	29.02	15.18	261.05	27.04	14.15	161.34	16.71	8.74
Dimension, ft									
A	0.89	2.76	3.83	0.93	2.88	4.01	1.05	3.25	4.51
B	0.44	1.38	1.91	0.46	1.44	2.00	0.52	1.62	2.25
C	1.40	4.36	6.05	1.28	3.98	5.52	1.25	3.90	5.41
D	1.38	4.30	5.96	1.38	4.28	5.94	1.78	5.52	7.66
E	0.69	2.15	2.98	0.69	2.14	2.97	1.09	3.38	4.68
F	1.89	5.88	8.16	1.97	6.12	8.49	3.22	10.00	13.87
G	0.63	1.96	2.72	2.43	7.57	10.49	2.09	6.50	9.01
H	2.94	9.14	12.68	1.97	6.12	8.49	0.52	1.62	2.25
I	1.78	5.54	7.69	0.99	3.06	4.25			
J	0.89	2.77	3.84	3.36	10.43	14.47			
K	3.21	9.97	13.83	3.88	12.05	16.70			
L	3.54	10.98	15.23						

[a]Assuming specific weight = 149.5 lb/ft^3.

Source: Shore Protection Manual, U.S. Army Corps of Engineers, Vicksburg, MS, 1984.

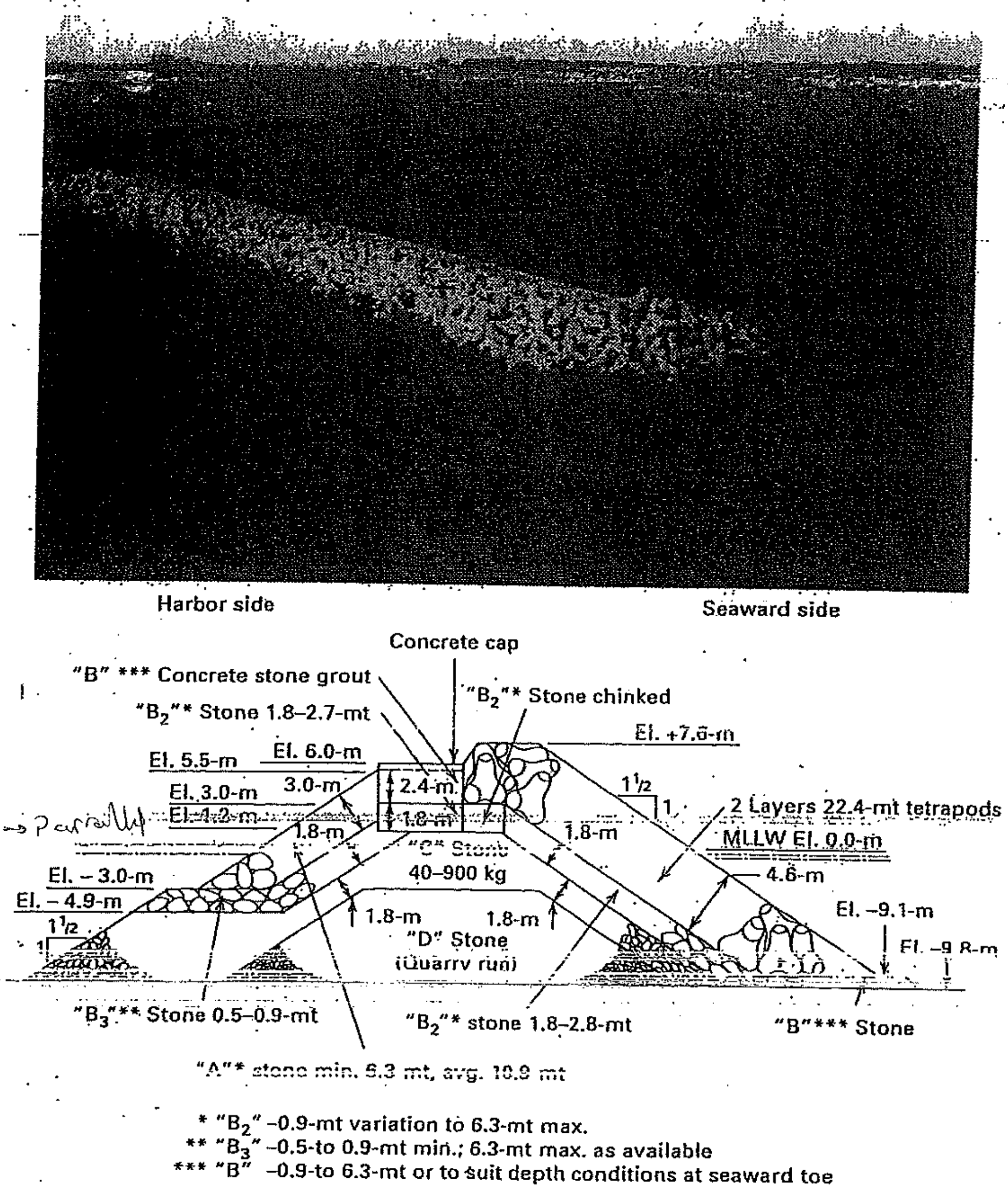


Figure 20-2 Tetrapod-rubble-mound breakwater at Crescent City, California. (Courtesy U.S. Army Corps of Engineers.)

structures do not have to be constructed to heights as great as those required for vertical-wall breakwaters.

Vertical-wall breakwaters include the following types:

1. Timber or precast concrete cribs filled with large stones
2. Concrete caissons filled with stone or sand
3. Sheet piling breakwaters

Types of Vertical B.W

Timber or precast concrete cribs consist of large boxlike compartments of open construction that are placed on a prepared foundation and then filled with stone. Concrete caissons are massive watertight boxes that are floated into position, settled on a prepared foundation, and filled with stone, earth, or sand. Usually, these caissons are then covered with a concrete slab.

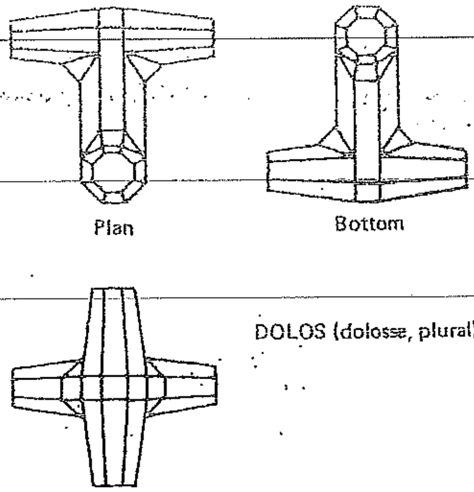


Figure 20-3 Views of dolos armor unit. (Source: Reference 4.)

The simplest sheet piling breakwater consists of a single row of piling that may or may not be strengthened by vertical piles. Another type of sheet piling breakwater consists of double walls of sheet piling connected by tie bars with the space between the walls filled with stone or sand. Finally, vertical-wall breakwaters may be built of cellular sheet pile structures that are filled with earth, stone, or sand to provide stability. A photograph of such a breakwater is shown in Fig. 20-4.

Of course, one can find breakwaters that are neither mound-type nor vertical-wall-type structures but that are composite structures containing features common to both broad classifications. Small vertical walls are often superimposed on top of rubble mound breakwaters, and it is not uncommon to find rubble mound foundations that support vertical-wall breakwaters.

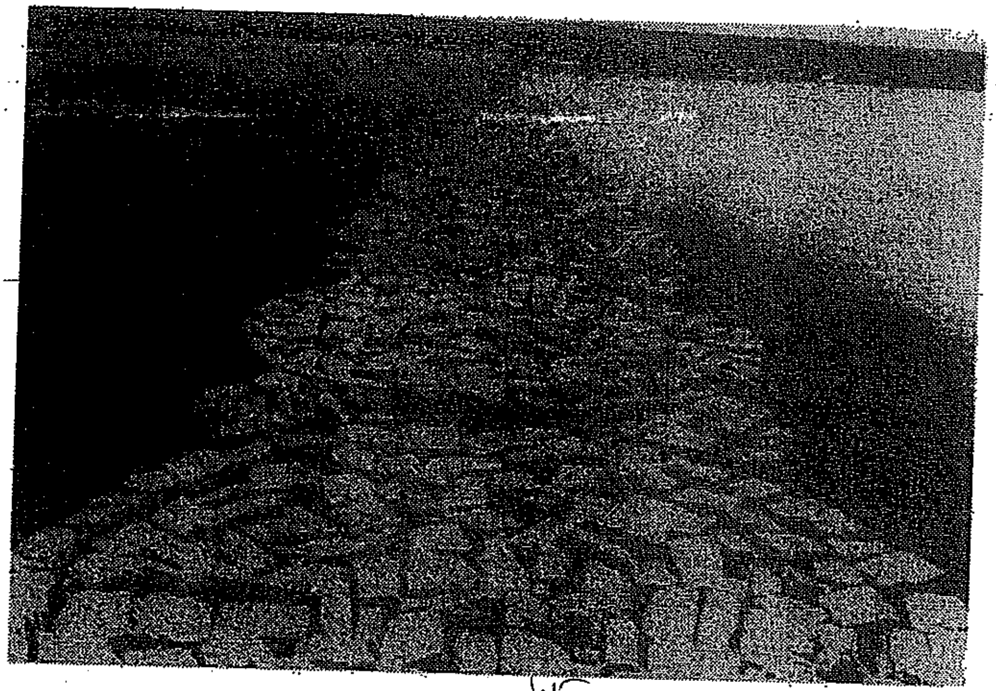
Several novel concepts in breakwater design have been studied, including floating breakwaters, hydraulic breakwaters, and bubble breakwaters. These proposals appear to have very limited usefulness. In the final analysis, the selection of breakwater type will depend on its purpose, foundation conditions, wave forces, availability of materials, and costs.

Breakwaters may not be connected to the shore and may be constructed as a single unit or as a series of relatively short structures. The latter approach may provide good protection from waves without the formation of undesirable sand shoals between the breakwater and the shore.

Height of B.W & width

The height of a breakwater will depend principally on maximum tide elevation, wave height, and breakwater type. Waves tend to break on mound structures, and the required height will vary with the angle of breakwater slope, wave height and length, and smoothness and permeability of the face of the structure. A wave will rise higher on a vertical-wall breakwater, and these structures should be built to an elevation equal to or greater than about 1.5 times the maximum wave height plus the elevation of maximum tide.

As a rule of thumb, the minimum width of the top of a mound breakwater should be approximately equal to the height of the maximum wave [5]. The width of the top and bottom of a vertical-wall breakwater should be determined by an analysis of the various



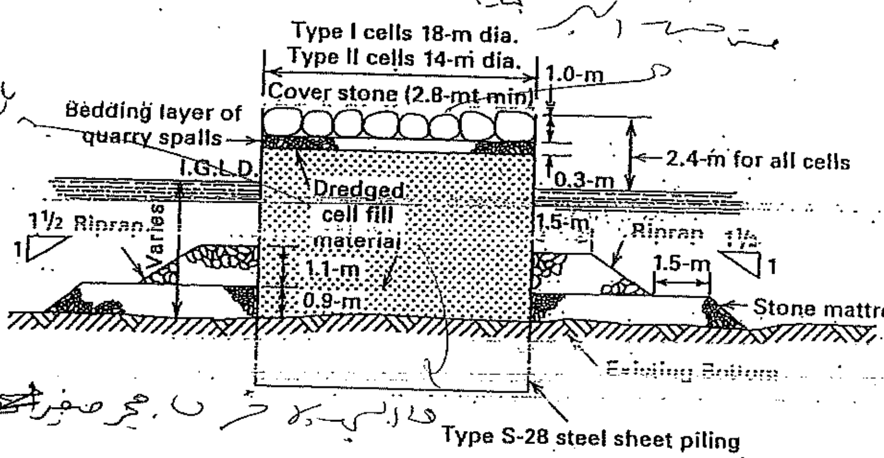


Figure 20-4 Cellular steel sheet-pile breakwater at Grand Marais Harbor, Michigan. (Courtesy U.S. Army Corps of Engineers.)

forces on the structure. These dimensions should be sufficient to prevent the structure from overturning, allowing a suitable factor of safety.

20-5. GROINS

The erosion of beach areas represents one of the most complex and difficult problems facing the coastal engineer. This erosion results from the effects of breaking waves, especially when the waves approach the shoreline at an oblique angle. Waves approaching a shoreline obliquely create a current that generally parallels the shoreline. This current, which is called a longshore current, sweeps along the sandy particles that constitute the beach bottom. The material that moves along the shore under the influence of waves and the longshore current is called littoral drift.

It is not always understood that man-made structures may stabilize one beach area while creating additional problems of beach erosion in adjoining areas. Impermeable seawalls, bulkheads, or revetments that are constructed along the foreshore often upset the delicate natural regime and cause or increase erosión immediately in front of the structure and along the unprotected coastline downdrift from the structure.

The most common approach to the control of beach erosion is to build a groin or a system of groins. A groin is a structure that is constructed approximately perpendicular to the shore in order to retard erosion of an existing beach or to build up the beach by trapping littoral drift. The groin serves as a partial dam that causes material to accumulate on the updrift or windward side. The decrease in the supply of material on the downdrift side causes the downdrift shore to recede. The shoreline changes that follow the construction of a properly designed groin system are shown in Fig. 20-5.

There are two broad classes of groins: permeable and impermeable. Permeable groins permit the passage of appreciable quantities of littoral drift through the structure. Impermeable groins, the most common type, serve as a virtual barrier to the passage of littoral drift.

A wide variety of groins has been constructed, utilizing timber, steel, concrete, and stone. A typical timber-steel sheet-pile groin is shown in Fig. 20-6. The type shown provides an impermeable barrier. Cellular steel sheet-pile groins also have been employed successfully, as have prestressed concrete sheet-pile groins and rubble mound groins. The latter usually is constructed with a core of fine quarry run stone to prevent the passage of sand through the structure.

The selection of the type of groin will depend to a large degree on the following factors:

1. Availability of materials
2. Foundation conditions
3. Presence or absence of marine borers
4. Topography of the beach and uplands

The hydraulic behavior of a system of groins is exceedingly complex and its performance will be influenced by:

1. The specific weight, shape, and size of the particles that constitute the littoral drift
2. The height, period, and angle of attack of approaching waves

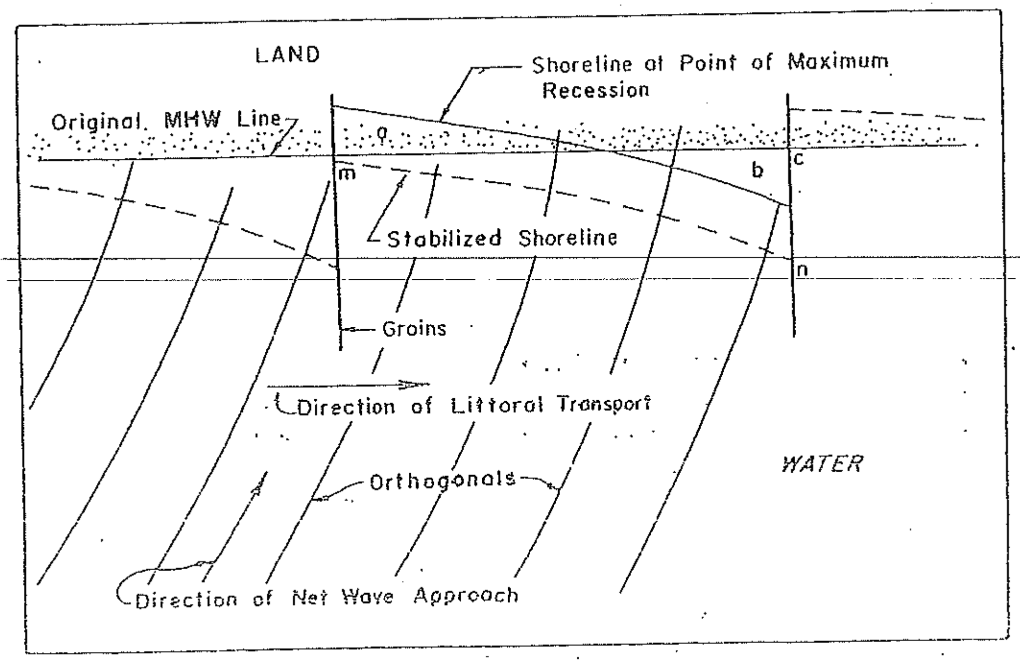


Figure 20-5 Groin system operation. (Courtesy U.S. Army Corps of Engineers.)

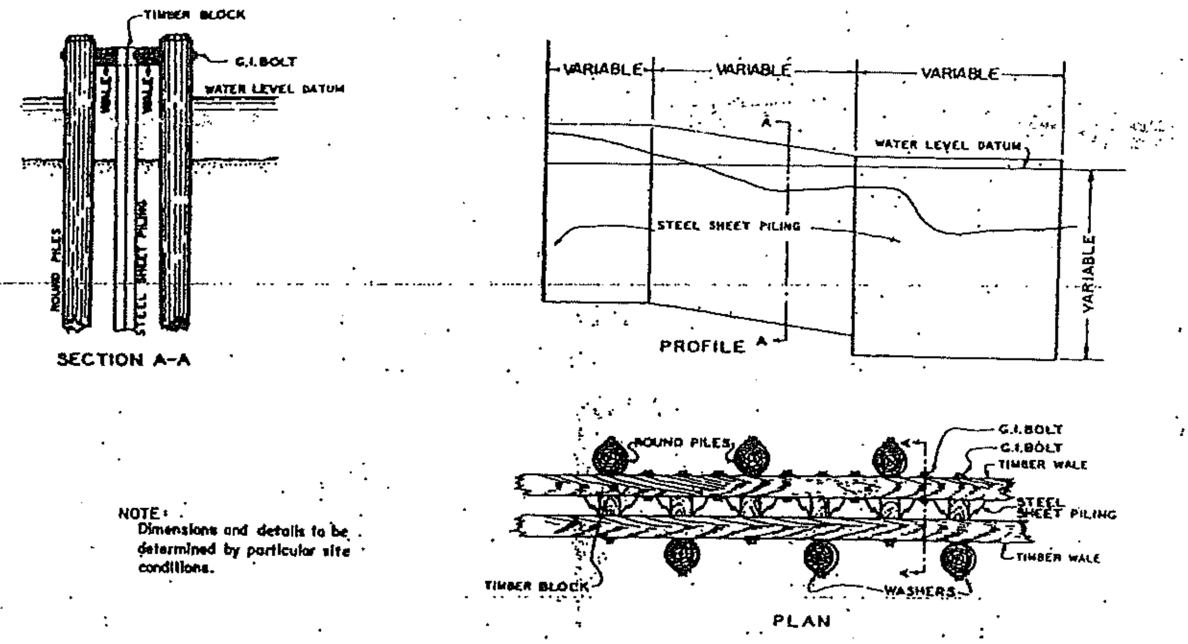


Figure 20-6 A typical timber-steel sheet-pile groin. (Courtesy U.S. Army Corps of Engineers.)

3. The range of the tide and the magnitude and direction of tidal currents
4. The design features of the groin system, including the groin orientation, length, spacing, and crown elevation

Groins usually are built in a straight line and are oriented approximately normal to the shoreline. There appears to be little advantage to the use of curved structures or groins of the T or L head types. These types tend to be more expensive to build and will normally experience more scour at the end of the structure than will be experienced with the straight groin.

There are no reliable analytical techniques for the determination of the desirable length of a groin. The length will depend on the nature and extent of the prevailing erosion and the desired shape and location of the stabilized shoreline. The total length, which typically is on the order of 30 to 45 m (100 to 150 ft), is comprised of three sections:

1. The horizontal shore section, which extends from the landward end to the desired location of the crest of the berm
2. The intermediate sloped section, which extends between the horizontal shore section and the outer section, roughly paralleling the slope of the foreshore
3. The outer section, which is horizontal

Spacing b/w groins

The spacing of groins in a groin system depends on the groin length and the desired alignment and location of the stabilized shoreline. As a rule of thumb, the U.S. Army Corps of Engineers [4] recommends that the spacing between groins should be equal to two or three times their length from the berm crest to the seaward end. Width

The elevation of the crest of a groin will determine to some extent the amount of sand trapped by the groin. In cases where it is desirable to maintain a supply of sand on the leeward side of the groin, it may be built to a low height, allowing certain waves to overtop the structure. If no passage of sand beyond the groin is desired, the elevation of the crest should be such that storm waves will not overtop the structure.

20-6. SEAWALLS, REVETMENTS, AND BULKHEADS

Seawalls, revetments, and bulkheads are structures that are placed parallel to the shoreline to separate the land from the water. While these structures have this general purpose in common, there are significant differences in specific function and design. Seawalls are massive structures that are usually placed along otherwise unprotected coasts to resist the force of waves. Seawalls are gravity structures that are subjected to the forces of waves on the seaward side and the active earth pressure on the shoreward side.

A revetment is also used to protect the shore from the erosive action of waves. It is essentially a protective pavement that is supported by an earth slope. A bulkhead is not intended to resist heavy waves but simply to serve as a retaining wall to prevent existing earth or fill from sliding into the sea.

Typical structural types of seawalls are shown in Fig. 20-7. These include a sloping wall, stepped-face wall, and a curved wall. The latter wall face may be either the non-reentrant type, which is essentially a vertical wall, or a reentrant type, which turns the wave back upon itself.

Sloping or vertical-faced seawalls offer the least resistance to wave overtopping. The amount of wave overtopping can be decreased substantially by the use of an armor block facing, such as tetrapods [6]. The stepped-face seawall is used under moderate wave conditions. It, too, may experience objectionable wave overtopping when subjected to heavy seas and high winds.

Under the most severe wave conditions, massive curved seawalls are used most frequently. For this type of structure, the use of a sheet pile cutoff wall at the toe of the seawall is recommended to reduce scour and undermining of the base. As a further precautionary measure to prevent scour, large rocks may be piled at the toe of the structure. Both of these features may be seen in Fig. 20-8.

There are two broad classes of revetments, rigid and flexible. The rigid type of revetment consists of a series of cast-in-place concrete slabs. In essence, this type of revetment

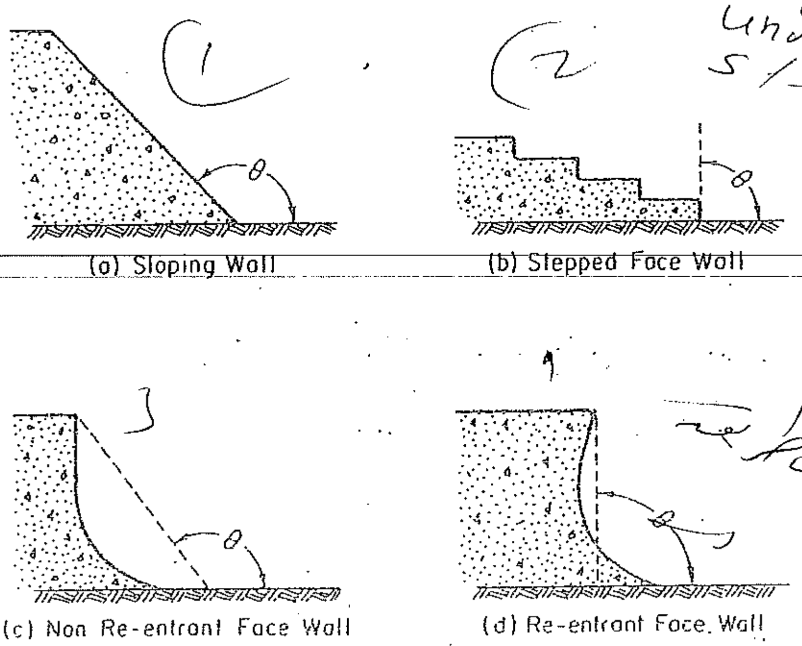


Figure 20-7 Typical sea walls.

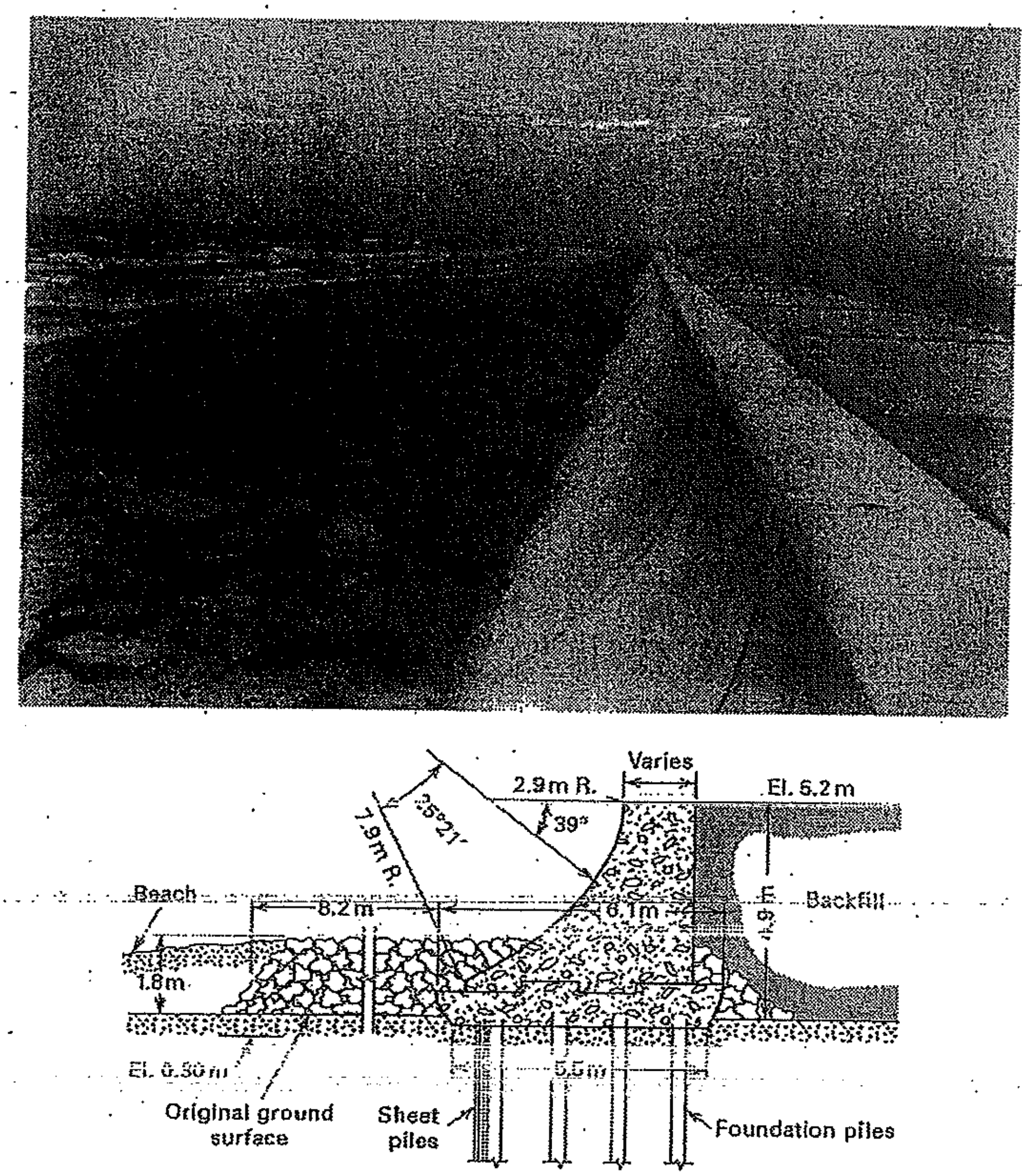


Figure 20-8 Concrete curved-face seawall at Galveston, Texas. (Courtesy U.S. Army Corps of Engineers.)

is a small sloping seawall.[3] Flexible or articulated armor-type revetments are constructed of riprap or interlocking concrete blocks that cover the shore slope. Typical riprap revetments have armor stones that weigh from 0.45 to 2.7 metric tons (½ to 3 tons). Concrete blocks used for revetments are commonly 0.45 to 1.2 m (1.5 to 4.0 ft) square and 50 to 300 mm (2 to 12 in.) thick. Sketches of typical revetments are shown in Fig. 20-9.

Two common types of bulkheads are illustrated in Fig. 20-10. A bulkhead is supported in a cantilever fashion by the soil into which it is driven. Additional support for a bulkhead may be provided by tie rods connected to vertical piles, which are driven some distance shoreward.

[3]The distinction between seawalls, revetments, and bulkheads is not sharp. A sloping faced seawall in one locality may be called a revetment in another. Similarly, a vertical-wall structure may be termed a bulkhead by some observers and a seawall by others.

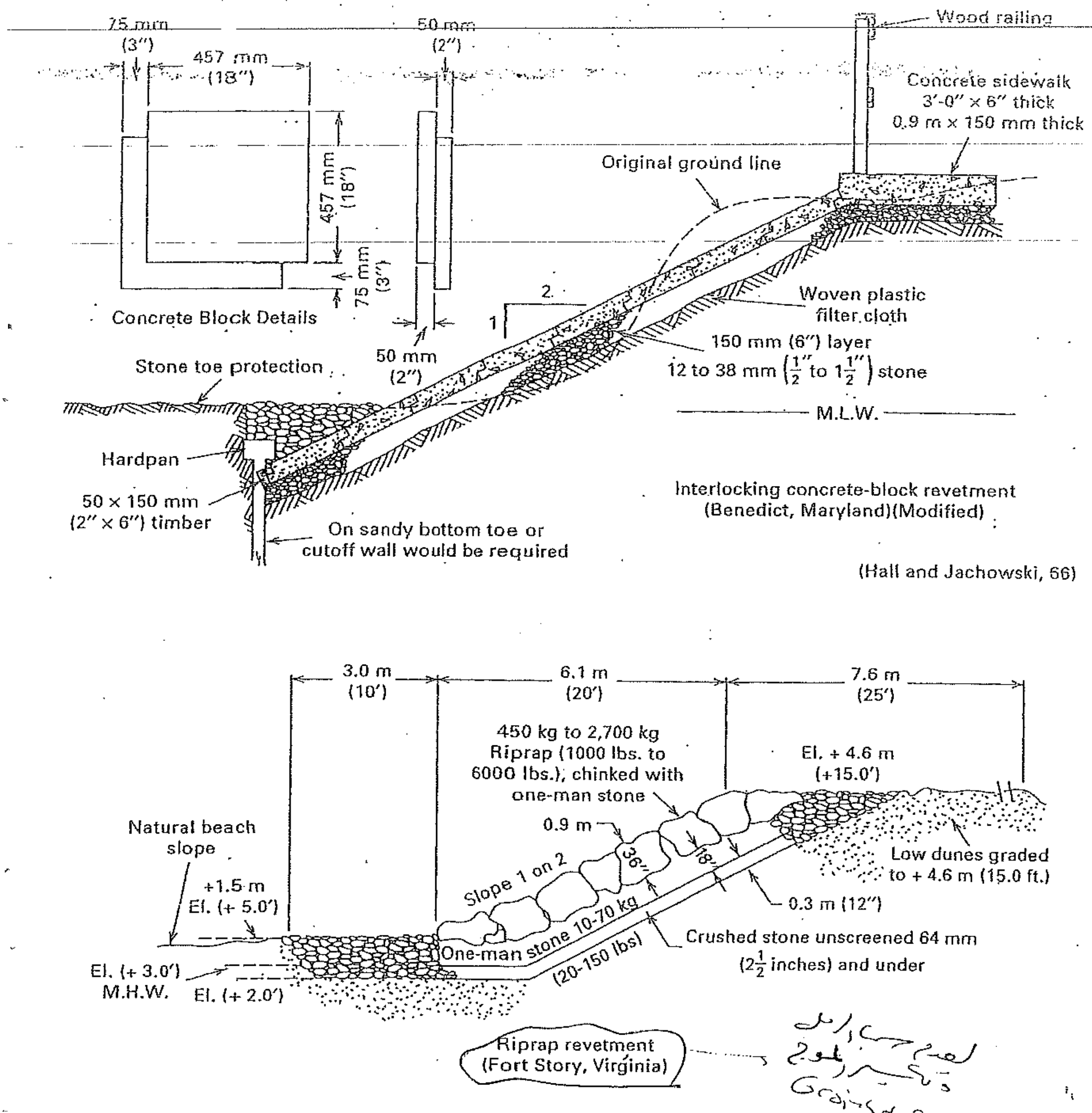
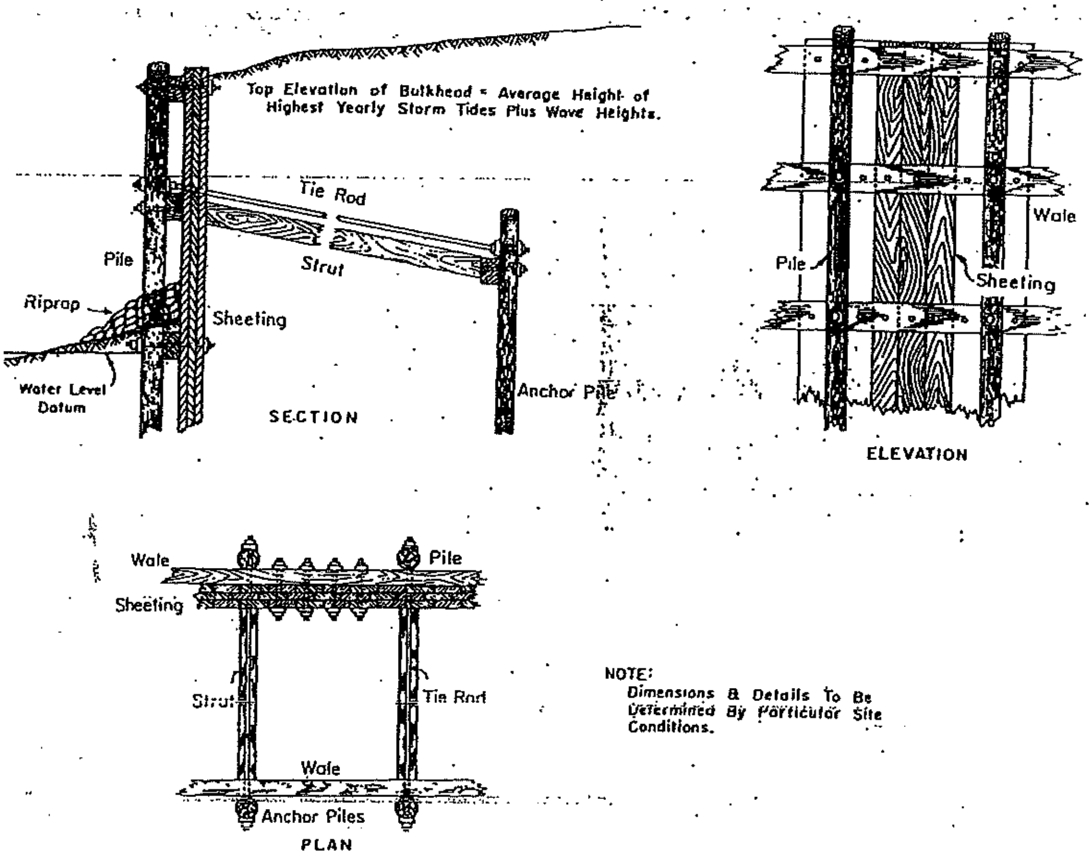


Figure 20-9 Typical flexible revetments. (Courtesy U.S. Army Corps of Engineers.)

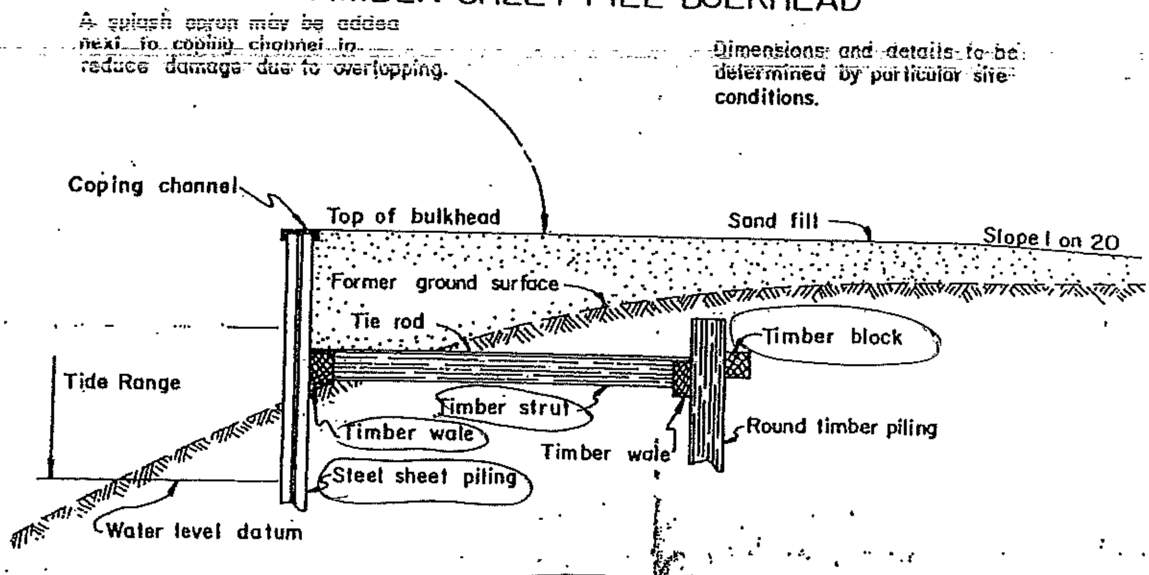
The choices of the location and length of seawalls, revetments, and bulkheads are usually straightforward depending as they do on local circumstances. The location of these structures with relation to the shoreline generally will coincide with the line of defense against further erosion and encroachment of the sea. The length of these structures depends on how much shoreline is to be separated from the water or protected from the sea.

Heights

A critical factor in the design of seawalls, revetments, and bulkheads is the determination of the height of the structure. This determination will hinge on a choice between two basic approaches to the problem. One approach is to design the structure so as to prevent wave overtopping, which might damage the structure, flood facilities on the landward side, and possibly endanger human lives. A second design approach is to recognize that it may not be feasible to construct the protective facility high enough to ensure that no wave overtopping will ever occur. This approach attempts to estimate the volume of water that



TIMBER SHEET PILE BULKHEAD



STEEL SHEET PILE BULKHEAD

Figure 20-10 Typical bulkheads. (Courtesy U.S. Army Corps of Engineers.)

under sheet structure

will pass over the top of the structure under the most critical wave conditions and to attempt to provide appropriate facilities to expel the overtopped water.

20-7. PROTECTIVE BEACHES

The *Shore Protection Manual* [4] describes the functions of protective beaches as follows:

> *Beaches can effectively dissipate wave energy and are classified as shore protection structures of adjacent uplands when maintained at proper dimensions. Existing beaches are part of the natural coastal system and their wave dissipation usually occurs without creating adverse environmental effects. Since most beach erosion problems occur when there is a deficiency in the natural supply of sand, the placement of borrow material on the shore should be considered as one shore stabilization measure. It is advisable to investigate the feasibility of mechanically or hydraulically placing sand directly on an eroding shore, termed* beach restoration, *to restore or form, and subsequently maintain, an adequate protective beach, and to consider other remedial measures as auxiliary to this solution. Also, it is important to remember that the replenishment of sand eroded from the beach does not in itself solve an ongoing erosion problem and that periodic replenishment will be required at a rate equal to natural losses caused by the erosion. Replenishment along an eroding beach segment can be achieved by stockpiling suitable beach material at its updrift end and allowing longshore processes to redistribute the material along the remaining beach. The establishment and periodic replenishment of such a stockpile is termed* artificial beach nourishment. *Artificial nourishment then maintains the shoreline at its restored position. When conditions are suitable for artificial nourishment, long reaches of shore may be protected at a cost relatively low compared to costs of other alternative protective structures.*

Reference 4 gives recommended procedures for:

1. Estimating the nourishment required to maintain stability of the shore
2. Sampling and testing native beach sand to obtain a standard for comparing the suitability of potential borrow sediments
3. Selecting borrow material
4. Determining an appropriate elevation, width, and slope of a beach berm
5. Treating the transition from the fill to the existing shoreline
6. Planning the proper location of a stockpile of nourishment material

20-8. SAND DUNES

Sand dune are also an important protective coastal formation. The line of dunes nearest the coast, known as foredunes, can prevent the movement of storm tides and waves into the land area behind the beach. Sand dunes near the beach may also serve as stockpiles to feed the beach. Dune ridges located farther inland also provide protection from waves but to a lesser degree than foredunes.

Reference 4 gives guidelines and suggestions for creation and stabilization of protective dunes by the use of slat-type snow fencing. Figure 20-11 illustrates sand accumulation by a series of four single-fence lifts along the Outer Banks, North Carolina.

Vegetation can also be employed to create stabilized dunes along the shore [4].

(Runup)

620

WAVE RUNUP AND OVERTOPPING

Where it is feasible to design a coastal structure to prevent wave overtopping, it is necessary to estimate the magnitude of the wave runup. Wave runup is defined as the vertical height above stillwater level to which water will rise on the face of the structure. Thus the runup, when added to the stillwater elevation, establishes the minimum elevation of the crest of the structure.

In recent years numerous laboratory investigations have been conducted to quantify the effects of several variables on runup. The research has shown that runup depends primarily on the structure shape and roughness, the water depth at the structure toe, the bottom slope in front of the structure, and certain wave characteristics.

The Corps of Engineers [7] has published a series of empirical curves, exemplified by Fig. 20-12, by which wave runup on smooth slopes can be estimated. Similar curves have been published for runup on slopes covered with riprap, rubble, and concrete armor units [8]. The curves are in dimensionless form giving the relative runup R/H_0' as a function of the deep-water wave steepness H_0'/gT^2 and structure slope, $\cot\theta$. Here R is the runup height measured vertically from the stillwater levels, H_0' is the unrefracted deep-water wave height, T is the wave period, and g is the acceleration of gravity.

Most of the studies of wave runup have been conducted using small-scale models. It has been found that there is a scale effect whereby the predicted values of wave runup from small-scale tests are smaller than those observed in practice. Table 20-2, which shows a scale correction factor as a function of the structure slope, indicates the magnitude of the scale effect.

620 Example

min elev (crest) of the structure = Runup + Stillwater elev

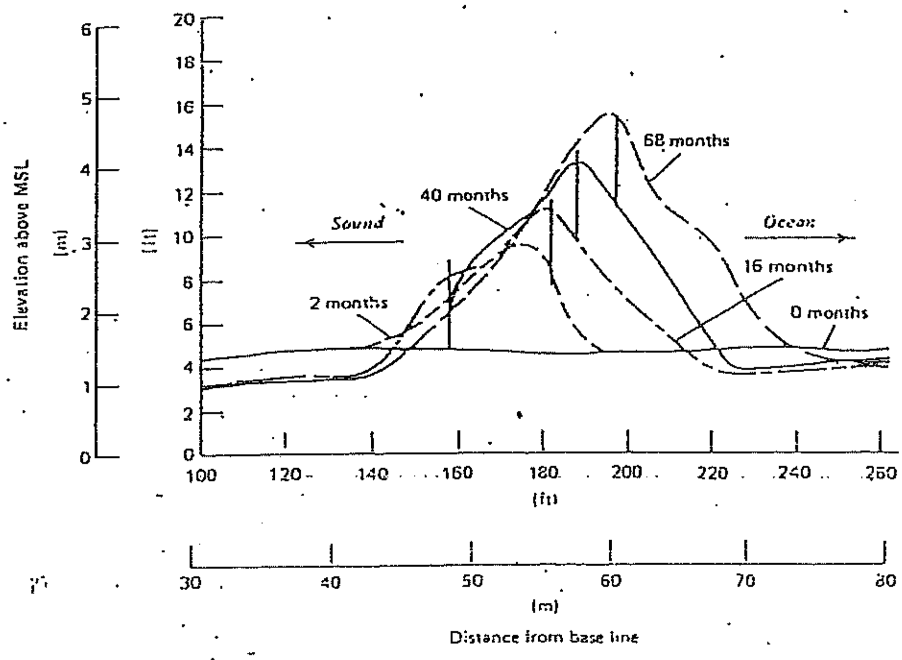


Figure 20-11 Sand accumulation by a series of four single-fence lifts, Outer Banks, North Carolina. (Source: Reference 4.)

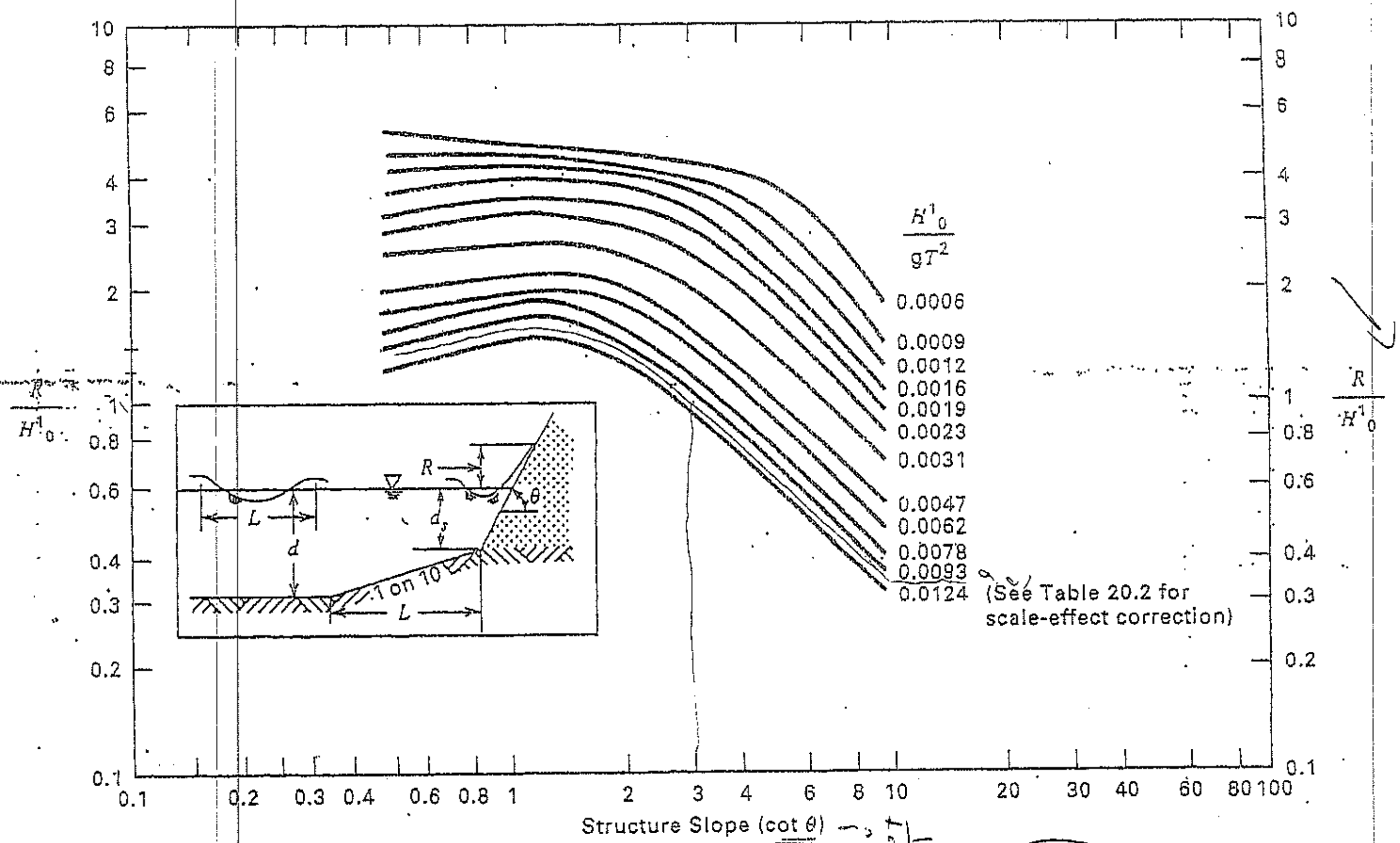


Figure 20-12 Relative wave runup curves for smooth slopes on 1 on 10 bottom; $d/L > 0.5$; $d_s/H'_0 = 0.6$ (Source: *Revised Wave Runup Curves for Smooth Slopes*, U.S. Army Corps of Engineers, July 1978.)

Table 20-2 Runup Scale Effect Correction Factors

Structure Slope, cot θ	Correction Factor, k
0.1	1.00
0.5	1.11
1.0	1.14
2.0	1.14
3.0	1.12
4.0	1.10
5.0	1.09
10.0	1.03

Source: Adapted from *Revised Wave Runup Curves for Smooth Slopes*, U.S. Army Corps of Engineers, Fort Belvoir, VA, July 1978.

The following example illustrates the use of Fig. 20-12 and similar figures.

Calculation of wall run up

EXAMPLE 20-1

Given: A design similar to that shown in Fig. 20-12 where a smooth, impermeable 1-on-3 structure is fronted by a 1-on-10 bottom slope. The depth at the toe of the structure is $d_s = 10$ ft, and the bottom slope extends seaward to a depth $d = 50$ ft. The deep-water wave height is 16 ft, and the wave period is 7 sec. Estimate the wave runup. The wave steepness is given as

1 الوحدات ft

$$\frac{H_0'}{gT^2} = \frac{16}{32.2(7)^2} = 0.0101$$

From Fig. 21-11

$$\frac{R}{H_0'} = 1.05$$

$$R = 16.8 \text{ ft}$$

From Table 20-2 the scale correction factor $K = 1:12$ and corrected $R = 16.8\ (1.12) = 18.8$ ft

Model studies have revealed that the volume rate of wave overtopping that occurs at a structure depends on the wave height and period, the water depth at the structure toe, and height, slope, type, and roughness of the structure. An empirical procedure for estimating the overtopping rate for various types of structures, water depths, and wave conditions is given by reference 5.

WAVE FORCES

Attempts by engineers to control the powerful and relentless forces of waves have often met with disappointment and failure. Rarely is an engineer called upon to design a structure to withstand forces of the magnitude of those imposed by ocean waves during a storm. Furthermore, wave phenomena are exceedingly complex, and reliable analytical equations for the estimate of wave forces generally are not available. Yet, by means of empirical data collected over a period of many years, the design of coastal and harbor structures can be approached with confidence by the application of sound engineering principles and procedures.

The magnitude of wave forces on coastal and harbor structures will vary a great deal depending on whether or not the waves break, the shape and slope of the face of the structure, and on the roughness and permeability of the structure.

The following material on wave forces is included for illustrative purposes only. For an in-depth treatment of this complex subject, the reader is referred to the *Shore Protection Manual* [4].

20-9. VERTICAL WALLS SUBJECTED TO NONBREAKING WAVES

$H_{clapotis} \approx 2 H_i$

Coastal structures located in protected areas or deep water may be subjected to nonbreaking waves. Observations have shown that when a nonbreaking wave strikes a vertical wall, the reflected wave augments the next oncoming wave and forms a standing wave, or *clapotis*. The height of a clapotis is approximately twice the height of the original wave.

Pressure distributions of the crest and trough of a nonbreaking wave at a vertical wall are shown in Fig. 20-13. When the crest of the clapotis is at the wall, the pressure increases from zero at the free water surface to $wd + P_1$ at the bottom. When the trough of the clapotis is at the wall, the pressure increases from zero at the free water surface to $wd - P_1$ at the bottom.

A method for estimating the wave forces from a nonbreaking wave was proposed by the Frenchman George Sainflou in 1928. With this method, the pressures due to nonbreaking waves are assumed to be essentially hydrostatic, and the resulting pressure distribution is approximated by a straight line.

The orbit center of the standing wave lies a distance h_o above the stillwater level. This distance may be computed by the following equation:

$$h_o = \frac{\pi H_i^2}{L} \coth\left(\frac{2\pi d}{L}\right) \tag{20-1}$$

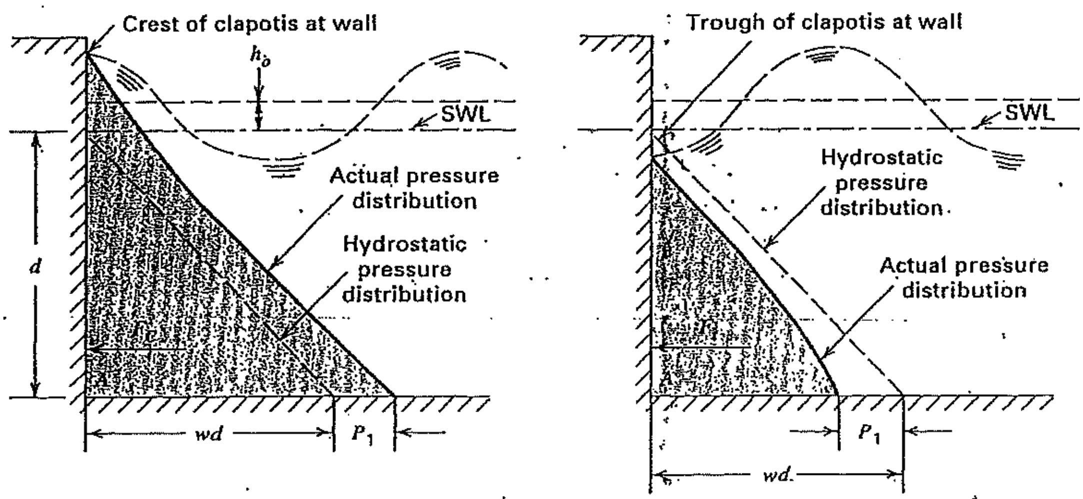


Figure 20-13 Pressure for nonbreaking waves. (Source: Reference 4.)

where

d = depth from stillwater level, m (ft)

H_i = height of the original free wave, or incidental wave, m (ft)

L = wavelength, m (ft)

In deep-water conditions, where the depth is more than one-half the wavelength, the equation reduces to

$$h_o = \frac{\pi H_i^2}{L} \tag{20-2}$$

The pressure P_1 is given by the equation

$$P_1 = \frac{wH_i}{\cosh\left(\dfrac{2\pi d}{L}\right)} \tag{20-3}$$

where

w = specific weight of water, 10 kN/m³ (64.0 lb/ft³) for salt water

When the same stillwater level exists on both sides of the wall, the Sainflou pressure diagram appears as Fig. 20-14. When the wave is at the crest position, the resultant pres-

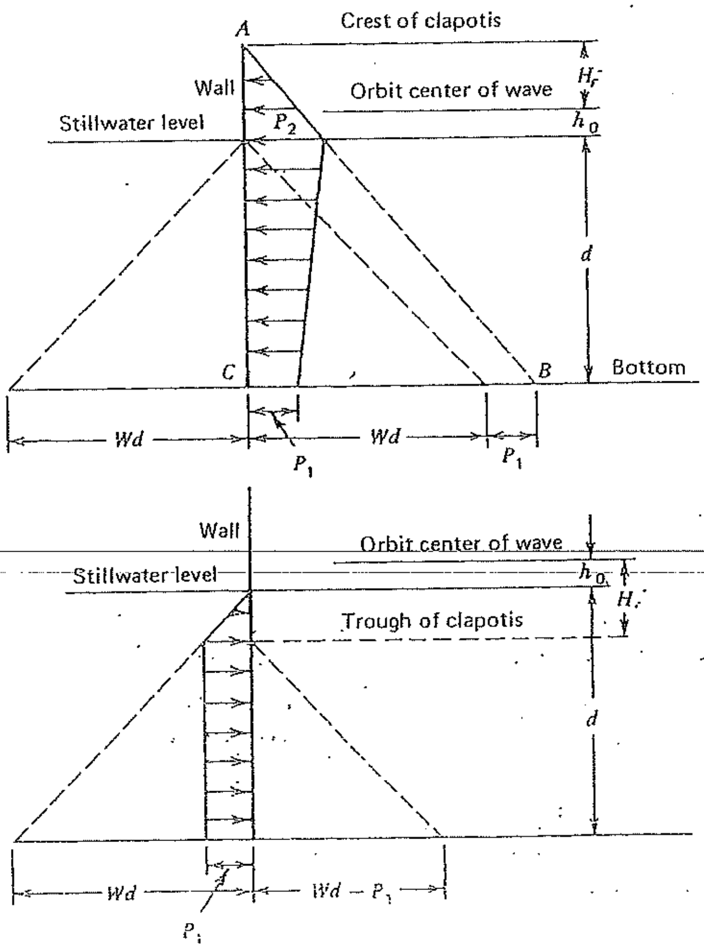


Figure 20-14 Wave pressures on vertical walls, according to Sainflou method.

sures are landward, as shown in Fig. 20-14*a*. When the trough of the wave is at the face of the wall, the net pressures are directed seaward, as shown in Fig. 20-14*b*.

By simple proportion, it can be seen that

$$P_2 = (wd + P_1)\left(\frac{H_i + h_o}{H_i + h_o + d}\right) \tag{20-4}$$

It is noted that if there is no water on the landward side of the wall, the resultant pressure with the crest at A would be that shown by the triangle ABC (Fig. 20-14*a*).

Experimental observations have indicated that the Sainflou method tends to overestimate forces due to steep, nonbreaking waves [4]. The method appears to be most suitable for smooth vertical walls where the wave reflection is most complete. Where wales, tiebacks, or other structural elements increase the surface roughness of the wall and retard the vertical motion of the water, a modification of the Sainflou method developed by Miche [9] and Rundgren [10] is recommended.

The Miche–Rundgren approach accounts for the fact that a wave may not be completely reflected to augment the next oncoming wave. That is, the height of the reflected wave, H_r, may be less than the height of the original free wave, H_i, and the *wave reflection coefficient* $\chi = H_r/H_i < 1.0$. With this approach, P_1 can be calculated by the equation

$$P_1 = \left(\frac{1 + \chi}{2}\right)\frac{wH_i}{\cosh(2\pi d/L)} \tag{20-5}$$

The Corps of Engineers [4] has published graphs to determine the height of the clapotis center above the stillwater level and the horizontal forces due to nonbreaking waves, exemplified by Figs. 20-15 and 20-16. The following example illustrates the use of the figures.

EXAMPLE 20-2

Given: A rough-faced vertical wall ($\chi = 0.9$) is to be built in water 4 m (13 ft) deep. The height of the original free wave, H_i, is 1.8 m (6 ft). The wave period is 6 sec. Estimate the nonbreaking wave forces against the wall resulting from the given wave conditions:

$$\frac{H_i}{gT^2} = \frac{1.8}{9.8(6)_2} = 0.005$$

$$\frac{H_i}{d} = \frac{1.8}{4.0} = 0.45$$

From Fig. 20-15,

$$\frac{h_o}{H_i} = 0.5 \qquad h_o = 0.9 \text{ m (3.0 ft)}$$

From Fig. 20-16,

$$\frac{F_c}{wd^2} = 0.54 \qquad \frac{F_t}{wd^2} = -0.32$$

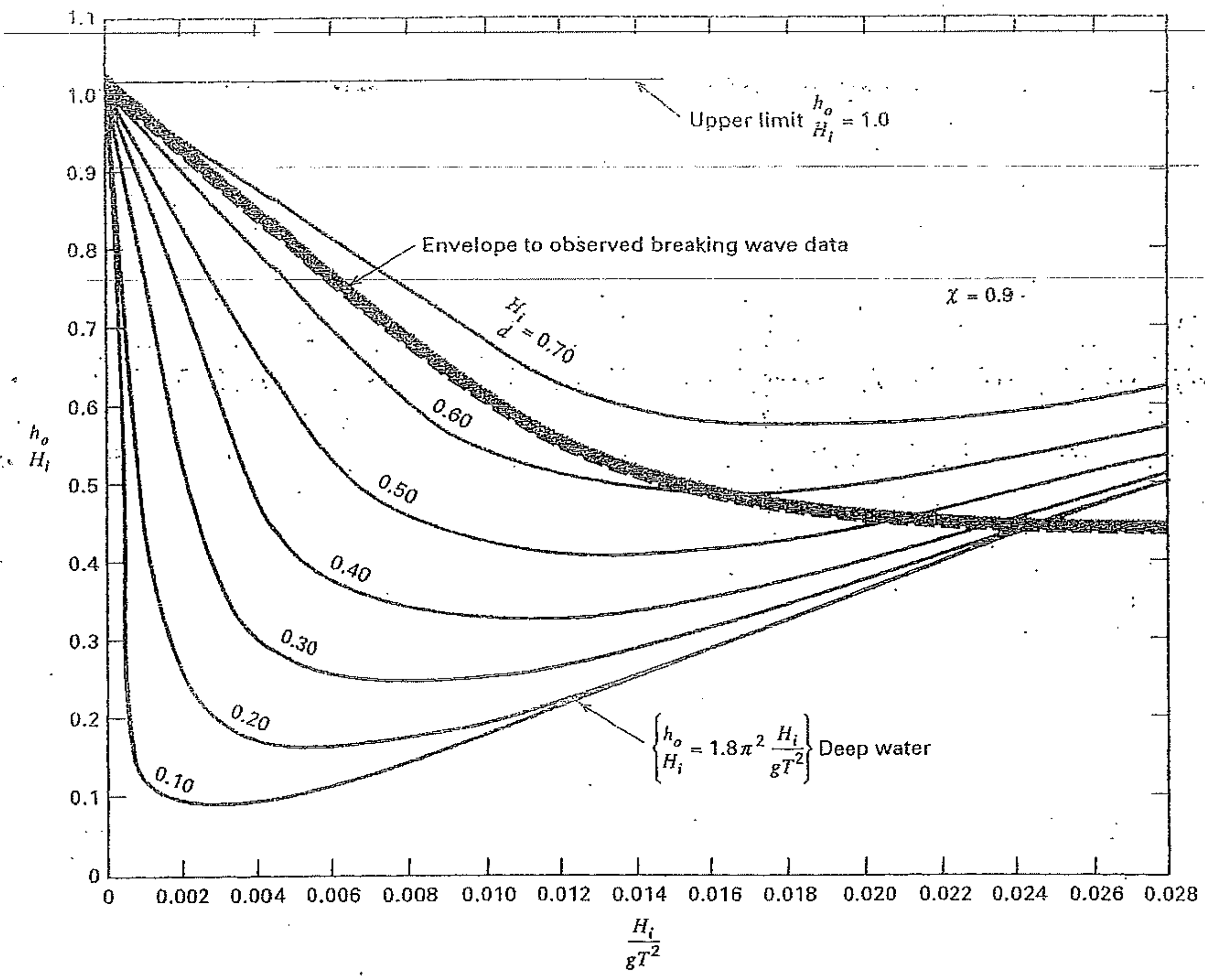


Figure 20-15 Dimensionless height of clapotis orbit center above stillwater level, nonbreaking waves, = 0.9. (Source: Reference 4.)

The force when the wave is at the crest is given as

$$F_c = 0.54\ (10)\ (4)^2 = 86.4 \text{ kN/m (5920 lb/ft)}$$

The force when the wave is at the trough is

$$F_t = -0.32\ (10)\ (4)^2 = -51.2 \text{ kN/m } (-3508 \text{ lb/ft})$$

$$F_c(\text{total}) = 86.4 + 0.5\ (10)\ (4)^2 = 166.4 \text{ kN/m (11,400 lb ft)}$$

$$F_t(\text{total}) = -51.2 + 0.5\ (10)\ (4)^2 = 28.8 \text{ kN/m (1,970 lb/ft)}$$

20-10. FORCES DUE TO BREAKING WAVES

Bulkheads, seawalls, and vertical-wall breakwaters often are located so as to be exposed to the force of breaking waves. Research has shown that these waves are much more complex than are nonbreaking waves. Model studies have indicated that structures exposed to breaking waves must withstand both hydrostatic and dynamic pressures. The dynamic pressure typically is intense and of short duration.

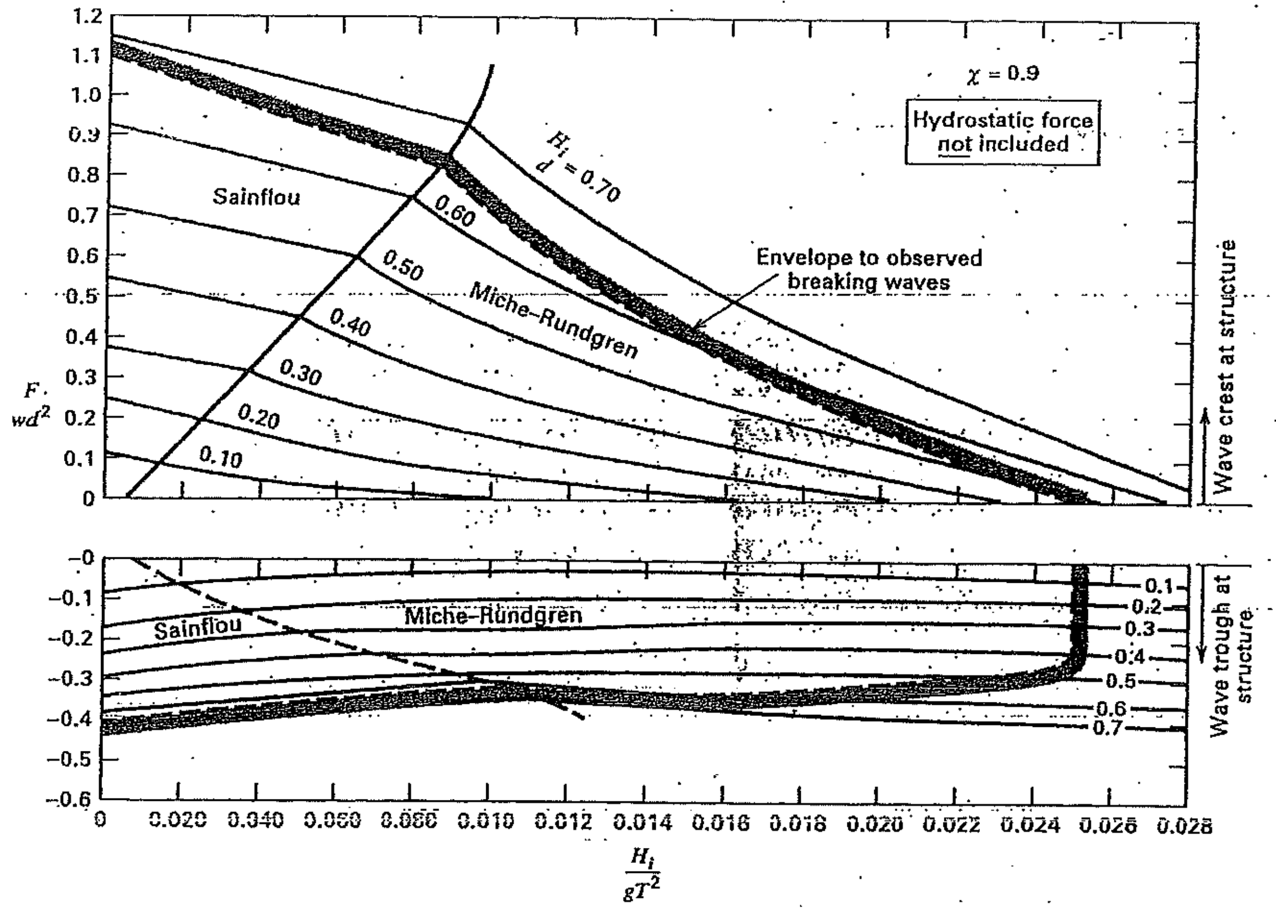


Figure 20-16 Dimensionless horizontal forces due to nonbreaking waves, = 0.9. (Source: Reference 4.)

The method commonly used to evaluate forces due to breaking waves was developed by R. R. Minikin [11].[4] The Minikin method was originally developed to analyze forces on a composite breakwater consisting of a concrete superstructure supported by a rubble mound substructure. According to this method, the dynamic pressure is assumed to be a maximum at stillwater level, decreasing parabolically to zero at a distance of one-half the wave height above and below the stillwater level. (See Fig. 20-17.)

The magnitude of the maximum dynamic pressure is given by

$$P_m = 101w\left(\frac{H}{L}\right)\frac{d}{D}(D+d) \qquad \text{kN/m}^2\ (\text{lb/ft}^2) \tag{20-6}$$

where

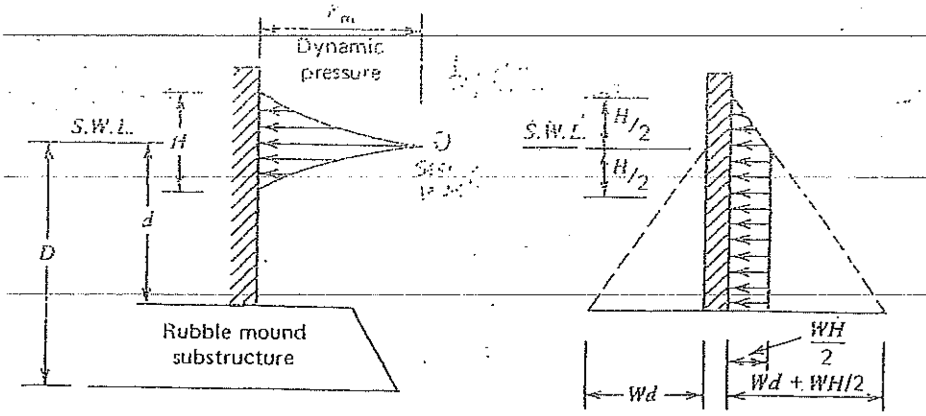
d = depth of water at the toe of the vertical wall, m (ft)
H = wave height at breaking, m (ft)

Other values in Eq. 20-6 are as defined previously.

The dynamic force per linear foot of structure is obtained from the area of the force diagram in Fig. 20-17*a*:

$$\text{Dynamic force per linear foot} = \tfrac{1}{3}P_m H \tag{20-7}$$

[4]An alternative method developed by Goda [12] is widely used in Japan. Comparisons of the Minikin and Goda methods have been published by Chu [13] and Ergin and Abdalla [14].



(a) Dynamic Pressure Diagram. (b) Hydrostatic Pressure Diagram.

Figure 20-17 Minikin wave pressure diagram.

In addition to the dynamic pressure, the Minikin method recognizes that ~~this~~ there is a hydrostatic pressure acting shoreward due to the height of the wave above stillwater level. (See Fig. 20-17*b*.) The magnitude of this force at the stillwater level is
pressure?

$$P_s = \tfrac{1}{2}wH \qquad (20\text{-}8)$$

Assuming that hydrostatic pressures exist on both sides of the wall, the net hydrostatic force per linear foot of structure is

$$\text{Hydrostatic force/linear m (ft)} = P_s d + \tfrac{1}{2}P_s(H/2) \qquad (20\text{-}9)$$

It follows that the resultant unit wave force per linear foot of structure is given as

$$R = \tfrac{1}{3}P_m H + P_s d + \tfrac{1}{4}P_s H \qquad (20\text{-}10)$$

An adaptation of the Minikin method has been used to calculate forces on caissons or walls that have no substructure. In this case, the values of D and L in Eq. 20-6 refer, respectively, to the water depth and wavelength measured one wavelength seaward from the structure. This is illustrated by Fig. 20-18.

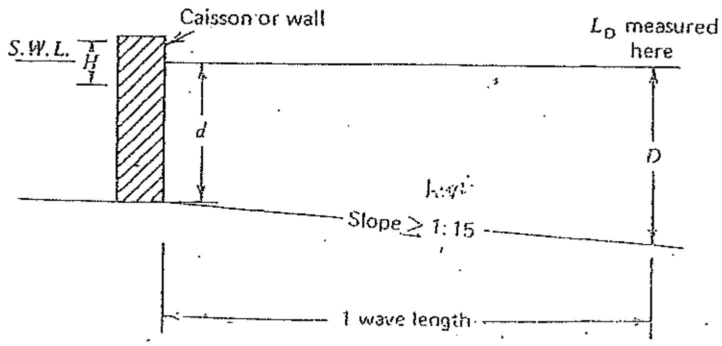


Figure 20-18 Parameters for adaptation of Minikin method

The preceding equation for dynamic wave pressures are applicable to vertical-wall faces and should give approximate values for stepped-faced structures. Walls that slope backward will experience smaller dynamic forces, and the Minikin method gives the following equation for the horizontal dynamic wave pressure:

$$P'_m = P_m \sin^2\theta \tag{20-11}$$

where
θ = the angle between the wall face and the horizontal

20-11. WAVE FORCES ON RUBBLE MOUND STRUCTURES

The stability of rubble mound structures depends on the ability of the individual armor units that comprise the armor layer to resist displacement. Thus, the designer's task is to determine the size of individual armor units required to withstand the attack of storm waves. Because of the complexity of the problem, it is necessary to rely on empirical equations that have been derived by means of extensive laboratory model tests as well as by observation of wall failures.

The following equation developed by the U.S. Army Corps of Engineers [4] is considered to be the most reliable guide for the estimation of the weight of each armor unit:

$$W = \frac{w_r H^3}{K_D (S_r - 1)^3 \cot\alpha} \tag{20-12}$$

where
W = weight of armor unit in primary cover layer, N (lb)
w_r = unit weight (saturated surface dry) of armor unit, kN/m^3 (lb/ft^3)
H = design wave height at the structure site, m (ft)
S_r = specific gravity of armor unit, relative to the water in which structure is situated,

$$S_r = \frac{w_r}{w_w}$$

w_w = unit weight of water, for seawater, 10 kN/m^3 (64.0 lb/ft^3)
α = angle of breakwater slope measured from horizontal, deg
K_D = an empirical constant (see Table 20-3).

This equation directly accounts for variations due to the wave height, slope of the structure face, and the unit weights of the water and the armor unit.

The empirical coefficient K_D is used to allow for the effect of the following variables:

1. Shape and roughness of the armor units
2. Number of units comprising the thickness of the armor layer
3. Permeability of the structure as affected by the placement of the armor units
4. Whether the structure is subjected to breaking or nonbreaking waves
5. Whether the armor unit is to cover the structure's trunk or head

Experience has shown that the head of a breakwater or jetty is more likely to sustain extensive damage than is the trunk. The provision of different values of K_D for the head and trunk allows for this fact.

It is noted that the values given in Table 20-3 provide little or no factor of safety.

Table 20-3 K_D Values for Use in Determining Armor Unit Weight, No-Damage Criteria

Armor Units	n[a]	Placement	Structure Trunk: Breaking Wave	Structure Trunk: Nonbreaking Wave	Structure Head: Breaking Wave	Structure Head: Nonbreaking Wave
Smooth rounded quarrystone	2	Random	1.2	2.4	1.1	1.9
Smooth rounded quarrystone	>3	Random	1.6	3.2	1.4	2.3
Rough angular quarrystone	2	Random	2.0	4.0	1.3–1.9[b]	2.3–3.2[b]
Rough angular quarrystone	>3	Random	2.2	4.5	2.1	4.2
Tetrapod	2	Random	7.0	8.0	3.5–5.0[b]	4.0–6.0[b]
Quadripod	2	Random	7.0	8.0	3.5–5.0[b]	4.0–6.0[b]
Tribar	2	Random	9.0	10.0	6.0–8.3[b]	6.5–9.0[b]
Tribar	1	Uniform	12.0	15.0	7.5	9.5
~~Dolos, slope = 2.0~~	~~2~~	~~Random~~	~~15.8~~	~~31.8~~	~~8.0~~	~~16.0~~
Dolos, slope = 3.0	2	Random	15.8	31.8	7.0	14.0

[a] n is the number of units comprising the thickness of the armor layer.

[b] K_D varies with the slope of the structure. Smaller values of K_D is applicable to slope, cot $\alpha = 3.0$. Larger values apply to cot $\alpha = 1.5$.

Source: Shore Protection Manual, U.S. Army Corps of Engineers, Vicksburg, MS, 1984.

PROBLEMS

1. Given: A smooth impermeable seawall has a 1-on-2 slope fronted by a 1-on-10 bottom slope similar to that shown in Fig. 20-12. The depth at the toe of the structure is 2.1 m, and the bottom slope extends seaward to the depth of 18 m. The deep-water wave height is 3.5 m, and the wave period is 8 sec. Estimate the wave runup.
2. Given: A smooth impermeable seawall has a 1-on-4 slope fronted by a 1-on-10 bottom slope similar to that shown in Fig. 20-12. The depth at the toe of the structure is 10 ft, and the bottom slope extends seaward to a depth of 50 ft. The deep-water wave height is 12 ft, and the wave period is 7 sec. Estimate the wave runup.
3. According to the Sainflou method, estimate the shoreward pressure on a vertical wall due to a nonbreaking wave that is 6.0 m in height and 60 m in length. The stillwater depth is 15 m. Assume the same stillwater exists on both sides of the wall.
4. According to the Sainflou method, estimate the shoreward pressure on a vertical wall due to a nonbreaking wave that is 18 ft in height and 220 ft in length. The stillwater depth is 40 ft. Assume that the same stillwater exists on both side of the wall.
5. Given: A rough-faced vertical wall ($\chi = 0.9$) is to be built in water 10.6 m deep. The height of the original free wave is 2.4 m. The wave period is 5 seconds. Estimate the nonbreaking wave forces against the wall resulting from the given wave conditions.
6. Given: A rough-faced vertical wall ($\chi = 0.9$) is to be built in water 15 ft deep. The height of the original free wave is 5 ft. The wave period is 7 sec. Estimate the nonbreaking wave forces against the wall resulting from the wave conditions.
7. A vertical wall is built on a bottom slope of 1:20. It is subjected to a breaking wave with the following conditions:

Wave height, H	2.8 m
Wave length at one wavelength from the structure, L_D	35 m
Stillwater depth to top of substructure, d	2.5 m
Depth to bottom at one wavelength from structure, D_D	3.9 m

Determine (a) the maximum dynamic pressure and the dynamic force per linear meter on the wall and (b) the hydrostatic force per linear meter.

8. A composite breakwater is subjected to a breaking wave with the following conditions:

Wave height, H	20 ft
Wave length, L	270 ft
Stillwater depth to top of substructure, d	40 ft
Depth to bottom, D	70 ft

Determine (a) the maximum dynamic pressure and dynamic force per linear foot on the breakwater and (b) the hydrostatic force per linear foot.

9. A vertical wall is built in deep water on a rubble mound substructure such as that shown in Fig. 20-17 and is subjected to a breaking wave. The wave height is 3.0 m, the depth to the toe of the substructure is 9.1 m, and the depth to the bottom is 12.2 m. The wave period is 6 sec. Estimate the maximum dynamic pressure and dynamic force per linear meter.
10. A rubble mound breakwater is to be built in saltwater using two layers of tetrapods with random placement as the armor unit. The structure slope is 1 (vertical) on 2 (horizontal). The weight of each armor unit is to be 14 metric tons, and its unit weight is 25 kN/m^3. What maximum wave height should this structure accommodate without significant damage?
11. A 33-metric-ton concrete armor unit is required for the protection of a rubble mound structure against a given wave height in saltwater. This weight was determined using a unit weight of concrete, $w_r = 22.8$ kN/m^3. What would be the required weight for concrete with $w_r = 22.0$ kN/m^3?

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Chapter 21

Planning and Design of Port Facilities

21-1. INTRODUCTION

Increasing world population, the industrialization of nations, and the reshaping of national boundaries have caused substantial increases in the volume of ocean commerce. The needs of increased world trade have resulted in increases in the number and size of the merchant fleet, which has created a growing need for the construction of new ports and the enlargement and modernization of existing port facilities.

In planning and designing port improvements, engineers must choose between alternate port layout schemes and substructure designs. They must obtain reliable forecasts of the number and time distribution of ship movements in order to anticipate the number of required berths. Adequate dimensions for channels and berths must be provided to allow safe and expeditious ship movements and berthing. Sufficient apron space also must be provided for the loading and unloading of ships and for the taking on of fuel and supplies.

General cargo terminals require properly designed transit sheds for sorting and temporary storage of packaged freight. Long-term storage facilities, both open and covered, usually are required, and special facilities may be needed for the handling and storage of chemicals, grains, and other materials in bulk.

Every attempt should be made in planning and designing a port to anticipate innovations and improvements in cargo-handling technology in order that prohibitively expensive alterations will not be required at a later date.

21-2. GENERAL LAYOUT AND DESIGN CONSIDERATIONS

In the early planning stages of the development of a port facility, certain basic decisions must be made regarding the general arrangement and layout of the facility. One such decision regards the choice of type of wharf.

A wharf is a structure built on the shore of a river, canal, or bay so that vessels may lie alongside and receive and discharge cargo and passengers. A wharf built generally parallel to the shoreline is called a marginal wharf or quay. A wharf built at an angle to the shore is called a finger pier or simply a pier. The berthing and maneuvering space between adjacent piers is called a slip.

Marginal wharves provide for easier berthing of ships and, therefore, generally are preferred over finger piers by steamship and stevedoring companies. Other advantages claimed for marginal wharves are:

1. The cost per berth may be lower.
2. Operational needs of ship owners, truckers, and stevedoring companies are satisfied better.
3. The costs of channel maintenance are less.
4. There is less hazard from waterfront fires and explosions.
5. The continuous line of wharf apron facilitates emergency movements of cargo between adjacent buildings or ships along the waterfront.

Piers, which generally favor rail operations, are preferred by railroad companies. The pier-type layout usually provides more berths per unit length of waterfront than does the marginal wharf arrangement.

The finger pier or slip type of construction may be obtained by extending the piers outward from the shoreline or by reshaping the shoreline by dredging the area between adjacent piers.

The choice of type of wharf layout will depend to a large extent on local conditions. At a port that has ample harbor area for ship maneuvering but that has a scarcity of waterfront land, constructing piers from the shore may prove advantageous. At a location where there is a scarcity of water area for ship movement and berthing, consideration should be given to dredging of slips. In the latter case, space must be provided for dumping the spoilage that is removed. Such excavation, of course, also removes from service land that otherwise could be devoted to on-shore port operations and storage.

deepen

21-3. PIER AND WHARF SUBSTRUCTURES

Basically, there are two broad classes of wharf substructures: (1) solid-fill type and (2) the open type. The solid-fill type, illustrated by Fig. 21-1, consists of a vertical wall that is backfilled by earth that supports a paved deck. The wall is commonly a cantilevered, anchored steel sheet-pile bulkhead. Gravity structures such as cellular steel bulkheads and cribs of timber or concrete also have been employed successfully.

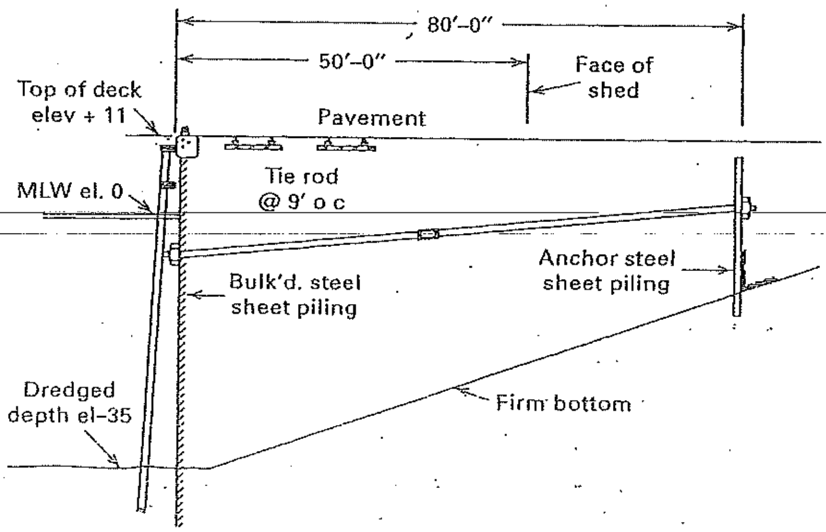


Figure 21-1 Solid fill type of wharf construction. (Courtesy The American Association of Port Authorities.)

In the open-type construction, the wharf superstructure is supported by timber, concrete, or steel piles. In this type of substructure, transverse rows of bearing piles are driven and capped with concrete girders. Longitudinal beams may also be provided to support unusually heavy concentrated loads. Alternatively, where heavy concentrated loads are not expected, the piles are spaced closely and capped with a flat slab. A typical open type of wharf substructure is shown in Fig. 21-2.

Solid adv.

The principal advantage of the solid-fill type of wharf substructure is that its great mass provides adequate resistance to the impact of mooring ships. Solid-fill substructures are inexpensive (except in deep water), are stable, and require little maintenance. Because this type of substructure serves as a barrier to currents and tides, it is used principally to support marginal wharves.

Open adv.

The open type of wharf substructure is more economical in deep-water locations and where a high-level superstructure is required. Since this type of substructure offers little restriction to water movements, it can be used to support piers in rivers and coastal areas alike.

Open substructures supported by timber piles are subject to decay and may be attacked by marine borers. Considerable risk of wharf fires is associated with the use of timber piles. These objections to the open type of substructure, of course, may be largely overcome by using concrete piles. Steel piles usually are encased in concrete above the low water line to inhibit corrosion.

A variation of the open-type wharf substructure utilizes a relieving platform on which fill is superimposed, capped by a paved deck. This type of design offers the advantages of high resistance to impact and economy of construction. High load concentrations are spread by the fill, lessening or eliminating the need for massive longitudinal girders. The relieving platform type of design is less subject to deterioration and decay than is the conventional open-type design, especially if the platform is located below the elevation of mean low water. A typical open-type wharf substructure with a relieving platform is shown in Fig. 21-3.

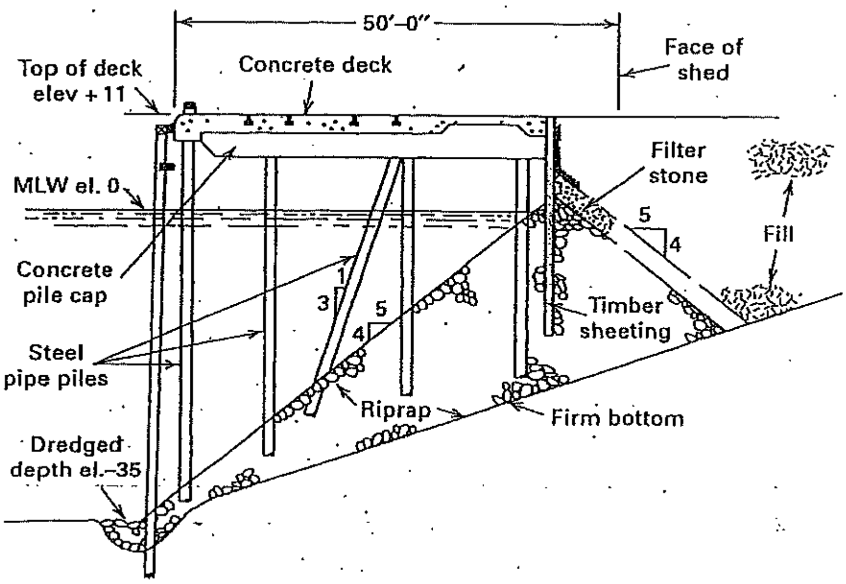


Figure 21-2 High-level open type of wharf construction. (Courtesy The American Association of Port Authorities.)

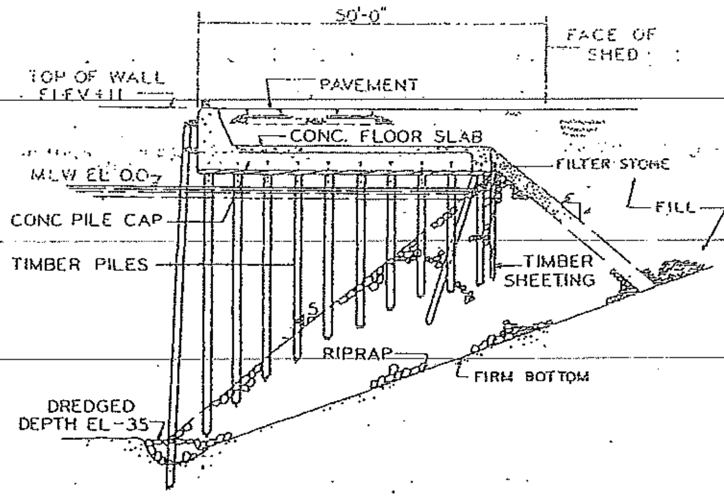


Figure 21-3 Open-type wharf substructure with concrete relieving platform. (Courtesy The American Association of Port Authorities.)

21-4. FENDER SYSTEMS

During the mooring process, a great deal of damage can be done to the wharf and ship unless some sort of protective device is provided to absorb the energy of the moving vessel. Such a device also is needed to lessen the effect of the bumping and rubbing of the ship against the wharf while the ship is secured. Protective installations that meet these needs are called fenders. A wide variety of fenders has been employed, including:

1. Pile fenders
2. Timber-hung systems
3. Rubber fenders
4. Gravity-type fender systems

One of the simplest fender systems involves a row of vertical wood piles that are driven on a slight batter and secured to the top edge of the wharf. By this system, impact is absorbed by deflection and by compression of the wood. A timber pile fender may be seen in Fig. 21-3. A floating log, called a camel, often is placed between the ship and the pile fenders to distribute impact loads along the fender system and keep the ship away from the face of the dock. The energy absorption of timber piles depends on the pile diameter and length and the type of wood [1]. It should be remembered that the energy absorption capabilities of wood piles, which are rather limited at best, decrease sharply with deterioration and wear.

Pile fenders constructed of steel and concrete also have been used, but generally these systems do not perform as satisfactorily as do timber piles.

In locations where the water is calm and the tidal range is small, a timber-hung fender system may be used. In this system, vertical wood members are secured to the face of the dock and terminate near the water surface. Typically, horizontal wood members are attached between the vertical members and the face of the dock. Timber-hung systems have a low energy absorption capacity that depends entirely on the compression of the wood.

Rubber has been used extensively and effectively in fender systems and in a variety of ways. Cylindrical or rectangular rubber blocks sometimes are used in wood fender systems to improve energy absorption capability. These blocks, which are placed behind horizontal members or vertical piles, absorb energy by compression.

Several patented rubber fender devices have been employed, including Raykin fender buffers and Lord fenders. Raykin fender buffers consist of layers of rubber cemented to steel plates and formed in a "V-shape." (See Fig. 21-4.) Energy is absorbed by this device as the rubber layers distort in response to the shearing forces imposed by a mooring vessel. Lord fenders consist of an arc-shaped rubber block bonded between two steel plates. Impact energy is absorbed by the bending and compression of an arc-shaped rubber column.

Hollow rubber cylinders have also been used as fenders. These vary in size from about 125 to 450 mm (5 to 18 in.) in outside diameter; the insider diameter is typically one-half the outside diameter. These cylinders are draped along the face of a wharf, suspended by a heavy chain. These fenders are suitable to protect a solid and deep wall such as the face of a relieving platform-type of substructure.

The energy absorption characteristics of rubber fenders are described in graphs and tables available from the rubber companies that sell these products.

Gravity-type fenders are made of large concrete blocks or cylinders that are suspended from the edge of the wharf deck. When a ship impacts the fender system, these heavy objects are lifted a short distance, absorbing the energy of impact. One such fender uses concrete-filled steel tubes that measure 0.6 to 0.9 m (2 to 3 ft) in diameter and 6 to 7.5 m (20 to 25 ft) in length. These cylinders, which weigh about 13.5 metric tons (15 tons), are suspended vertically along the side of the wharf and are hinged in such a way so as to be lifted when a lateral impact force is applied. Wood rubbing strips usually are attached to the seaward side of these cylinders. gravity

In the preceding paragraphs, a brief summary description of some of the more popular fender systems has been given. Table 21-1 lists the principal advantages and disadvantages of these systems.

In selecting a type of fender system the designer must consider a variety of factors, but the most important factors are:

1. Mass of ships to be berthed
2. Speed of berthing (normal to the dock)
3. Environmental conditions at the port

Selecting

The speed of approach normal to the dock taken for design purposes varies from about 0.03 to 0.4 m/sec (0.1 to 1.25 ft/sec), depending principally on the ship size, exposure of the wharf to wind, waves, tides and currents, and availability and type of docking assistance. Some guidance on selection of docking speed has been published by Lee [1].

The kinetic energy of a docking ship, expressed in joules (foot pounds), is given by the equation

$$E = \tfrac{1}{2} M v^2 \tag{21-1}$$

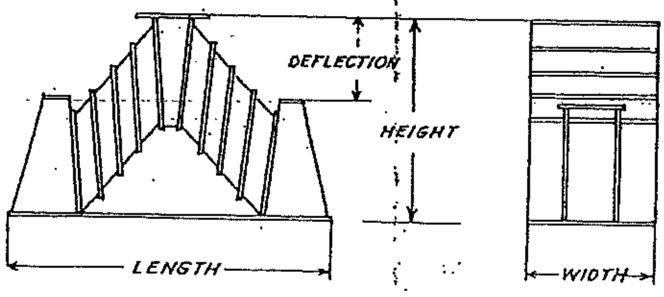


Figure 21-4 The Raykin fender buffer.

Table 21-1 Comparison of Various Types of Fender Systems

Fender System	Advantages	Disadvantages
Standard pile, timber	Low initial cost; timber piles are abundant in United States and most world regions	Energy absorption capacity limited and declines as result of biodeterioration; susceptible to mechanical damage and biological deterioration; high maintenance cost of damage and deterioration significant
Standard pile, steel	High strength; feasible for difficult seafloor conditions	Vulnerability to corrosion; high cost
Standard pile, reinforced concrete	Insignificant effects of biodeterioration	Energy absorption capacity very limited; corrosion of steel reinforcement through cracks
Timber-hung system	Very low initial cost; less biodeterioration hazard	Low energy absorption capacity; unsuitability for locations with significant tide and current effects
Rubber fender systems: rubber-in-compression	Simplicity and adaptability; effectiveness at reasonable cost	High concentrated loading may result; frictional force may be developed if rubber fenders contact ship hull directly; higher initial cost than standard pile system without resilient units
Rubber fender systems: rubber-in-shear	Capable of cushioning berthing impact from lateral, longitudinal, and vertical directions; most suitable for dock-corner protection; high energy-absorbing capacity for serving large ships of relatively uniform size; favorable initial cost for very heavy duty piers	Raykin buffers tend to be too stiff for small vessels and for moored ships subject to wave and surge action; steel plates subject to corrosion; bond between steel plate and rubber is a problem; high initial cost for general cargo berths

Table 21-1 *Continued*

Fender System	Advantages	Disadvantages
Gravity-type fender systems	Smooth resistance to impacts induced by moored ships under severe wave and swell action; high energy absorption and low terminal load can be achieved through long travel for locations where excessive distance between ship and dock is not a problem	Heavy berthing structure required; heavy equipment required for installation and replacement; high initial and maintenance costs; excessive distance between dock and ship caused by the gravity fender is undesirable for general cargo piers and wharves

Source: T. T. Lee, "Design Criteria Recommended for Marine Fender Systems," *Proceedings of Eleventh Conference on Coastal Engineering,* American Society of Civil Engineers, New York, September 1968.

where

M = mass of ship, kg (slugs)

v = berthing velocity normal to face of the dock, m/sec² (ft/sec²)

For ships docking at moderate to high speeds, the kinetic energy is increased due to the mass of water moving alongside the ship. To allow for this effect, the mass value used in Eq. 21-1 should be increased by about 60 percent. $M + 0.6M = M'$

For planning purposes, it may be assumed that one-half of the kinetic energy is to be absorbed by the fender system.

The load deflection characteristics of various fender systems is shown in Fig. 21-5.

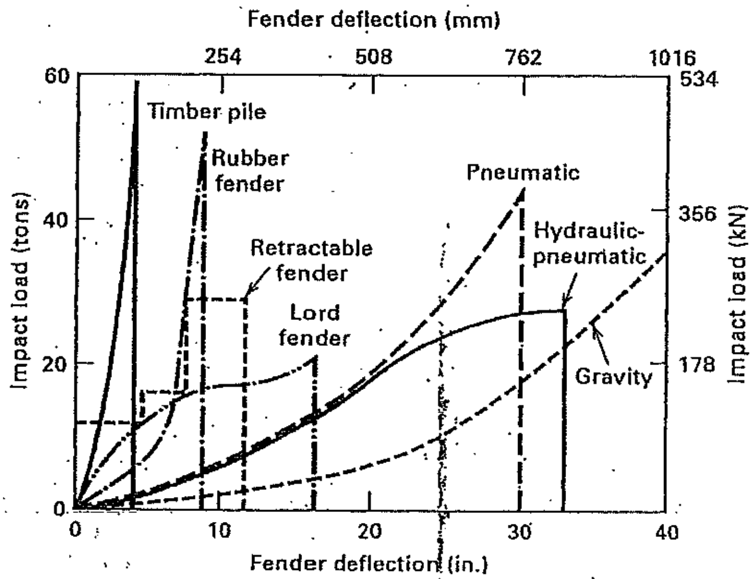


Figure 21-5 Load-deflection characteristics of various fender systems. (Source: Theodore T. Lee, "Design Criteria Recommended for Marine Fender Systems," *Proceedings of Eleventh Conference on Coastal Engineering,* September 1968.)

21-5. CLASSIFICATION OF BERTHS BY CARGO TYPE

The layout and design of port facilities vary significantly by type of cargo handled in size, type of loading and unloading and storage facilities, as well as types of transport facilities for moving the cargo to inland points.

Cargo handled at seaport berths may be classified as follows [2]:

Break-bulk general cargo includes various noncontainerized goods and commodities in small packages, bags, or boxes that are usually loaded and unloaded on small pallets by ship's gear.

Neo-bulk general cargo consists of noncontainerized miscellaneous goods and commodities shipped, packaged, and transported as units. Examples of neo-bulk general cargo include automobiles, lumber in stacks, and heavy machinery.

Containerized general cargo is miscellaneous goods and commodities shipped in standard-sized large boxes called containers such that only the container is handled in transit through the terminal.

Dry bulk cargo consists of dry granular materials that are transported in the bulk state, including grain, which must be stored in covered buildings or silos, and metallic ores, which may be stored in open piles.

Liquid bulk cargo includes liquid commodities that are not packaged in barrels, drums, or similar containers. The largest commodity in this category is crude oil or finished petroleum products. The category also includes edible oils, molasses, and liquid chemicals.

Approximately 2.6 percent of the berths at U.S. seaport terminals are for passenger ships or ferries. About one in six berths at U.S. seaports are for barges, for mooring, or inactive [3].

21-6. PORT PLANNING

The planning of the size of a port begins with an appraisal of the volume of present and future commerce and types of shipping. This essential information allows the port planner to estimate the number, type, and sizes of the ships to be accommodated. The forecasting of the anticipated volume and type of freight is an area of concern of economists and planners, and engineers seldom are involved in this type of activity. While the detailed procedures for making such a study do not fall within the purview of this chapter, a brief general discussion of certain aspects of this problem will be given.

Commerce

The commerce to be shipped through a proposed port will depend most of all on the nature and size of its tributary area, or hinterland. The hinterland generally is defined as that area within which the overall cost of freight movements through the port in question is equal to or less than corresponding costs via competing ports, based on existing rates and charges.

The forecasting of freight movements for a proposed port normally would include an evaluation of existing and future levels of activity in manufacturing, mining, oil drilling, agriculture, and forestry within the hinterland. Significant changes in berthing, cargo handling, and storage capabilities of competing ports should also be considered carefully. When expansion of existing port facilities is contemplated, the current level of port activity and its efficiency in handling current traffic are considerations of foremost concern.

Another important aspect of port planning involves estimating the capacity of a port, usually expressed in terms of throughput in metric tons (or tons) per year per terminal. The annual cargo projection of metric tonnage (or tonnage) by cargo class can then be divided by the corresponding capacity to determine how many terminals of a given cargo class would be required.

Table 21-2 Typical Annual Cargo Throughput for One-Berth Modules

Single-Berth Terminal By Cargo Class	Annual Cargo Throughput	
	Metric Tons	Short Tons
Break-bulk general cargo	59,875	66,000
Neo-bulk general cargo		
Low density (autos, lumber, etc.)	163,300	180,000
High density (steel)	362,875	400,000
Containerized general cargo		
Single berth	1,225,000	1,350,000
Two or more berths (per berth)	1,500,000	1,650,000
Dry-bulk		
Silo storage	907,000	1,000,000
Open storage, low density	453,500	500,000
Open storage, high density	907,000	1,000,000
Liquid bulk, other than petroleum	72,575	80,000
Petroleum bulk, up to 50,000 dwt[a] ships	1,360,000	1,500,000
Petroleum bulk, 50,000 to 200,000 dwt[a] ships	5,443,000	6,000,000

[a]dead weight ton, a measure of cargo capacity of a vessel, traditionally measured in long tons (2,240 lb).

Source: Port Handbook for Estimating Marine Terminal Cargo Handling Capability, U.S. Department of Transportation, Maritime Administration, Washington, DC, November 1986.

The *Port Handbook* [2] provides estimates of typical annual cargo throughput for one berth "modules." (See Table 21-2.) These estimates of throughput are practical working capabilities under ordinary operating constraints rather than intrinsic capacities. They are based on the results of extensive surveys of more than 30 U.S. ports. Each throughput estimate is for a single berth for a stated cargo class and assumes specific data for six throughput components; those for:

1. Ship characteristics
2. Cargo transfer at apron
3. Apron-to-storage transfer
4. Terminal storage capacity
5. Inland cargo transfer
6. Inland transfer processing

Table 21-3 shows the assumed values for each transport component for a break-bulk cargo terminal with covered storage.

The *Port Handbook* [2] allows the user to employ modifiers in the form of graphs to obtain a more refined throughput estimate where certain of the given data about the physical plant and operating characteristics of a given terminal differs from that assumed.

Another approach for estimating port capacity is the berth occupancy concept. In this approach, the operating capacity of a port is simply the product of the cargo-handling rate, in metric tons (or tons) per day per occupied berth and the number and extent and utilization of berths.

The rate of loading and discharging of cargo depends on:

1. Types of cargo
2. Vessel type and size (e.g., number of hatches)
3. Availability and size of stevedore gangs
4. Degree of mechanization and methods of cargo handling

Table 21-3 Break-Bulk General Cargo Terminal Module Covered Storage Variant

Subelement Values by Component
Ship characteristics
Maximum length: 560 ft
Maximum draft: 32 ft
Maximum size: 18,000 dwt[a]
Typical size: 15,000 dwt[a]
Typical shipload one way: 1300 tons
Typical cargo transfer per ship visit: 2000 tons/ship call
Typical time at berth: 2 days
Interarrival time: 11 days (33 ships)
Cargo transfer at apron
Type of transfer unit: longshoremen gang using ship's gear and forklifts
Number of units: 6
Typical transfer rate per unit: 14 tons/hr
Time to load/unload ship: 24 hr (working time)
Apron/storage transfer
Type of transfer unit: forklift trucks
Number of units: 12
Typical transfer rate per unit: 7 tons/hr
Time to transfer average shipload to/from storage: 24 hr
Terminal storage capacity
Gross backland area: 6.0 acres (261,000 ft^2)
Storage area: 2.7 acres
Auxiliary area: 3.3 acres
Yard storage capacity: 63,000 tons
Typical length of time a ton of cargo stays in storage: 13 days
Throughput density: 0.25 tons/ft^2/year
Inland cargo transfer
Type of transfer unit: truck loading dock spaces
Number of units: 10
Transfer rate per unit: 17 tons/hr
Time to load/unload a transport unit: 1 hr
Transport unit load: 17 tons
Typical daily cargo: 154 tons
Inland transfer processing
Type of transport unit: highway trucks
Peak units per day: 18
Gate processing time: 12 min
Gate processing capability: 5 units/hr
Number of processors: 1
Transport queue space: 42,000 ft^2

Source: Port Handbook for Estimating Marine Terminal Cargo Handling Capability, U.S. Department of Transportation, Maritime Administration, Washington, DC, November 1986.

[a]dead weight ton

Typical loading/unloading rates for various terminal classes are given in Table 21-4. In using these or other cargo-handling rates to estimate annual berth capacities, it should be remembered that a given berth will not be occupied 100 percent of the time during the year.

The estimation of the number of berths required must be made in the face of fluctuations in demand. In cold climates, allowance must be made for the closing of the port in winter months due to the formation of ice. Consideration may also have to be given to seasonal fluctuations in transportation of certain products.

Independent studies by Plumlee [4], Nicolaou [5], and Fratar et al. [6] have shown that ships arrive at a public seaport in accordance with a random pattern and that the Poisson probability distribution may be used to predict the number of days on which a particular number of ships will be present.

According to the Poisson law,

$$F_n = \frac{T(\bar{n})^n e^{-\bar{n}}}{n!} \tag{21-2}$$

Table 21-4 Typical Ship/Apron Cargo Transfer Rates Per 8-Hour Day

Cargo Class	Metric Tons/Day	Tons/Day
Break-bulk cargo terminals:		
using ship's gear with 6 gangs of longshoremen of 15 men	610	672
Neo-bulk cargo terminals		
Autos—self-propelled and roll-on, roll-off	725	800
Other low-density: using ship's gear and forklifts with 6 gangs of longshoremen of 15 men	610	672
Steel: using 4 shoreside cranes and ship's gear	1300	1440
Container terminals:		
Using 2 gantry cranes	2900	3200
Dry bulk terminals[a]		
Silo storage	2720	3000
Open storage, low density		
Loading	2250	2480
Unloading	725	800
Open storage, high density		
Loading	3265	3600
Unloading	1090	1200
Bulk petroleum terminals		
Maximum 50,000 dwt[b] ships using 4 loading arms	1960	2160
Maximum 50,000 to 200,000 dwt[b] ships using 6 loading arms	5960	6160

[a]Using 2 conveyor belt ship loaders/unloaders.

[b]dead weight ton.

Source: Port Handbook for Estimating Marine Terminal Cargo Handling Capability, U.S. Department of Transportation, Maritime Administration, Washington, DC, November 1986.

where

F_n = number of units of time that n ships are present during T time units

$\bar{n}$ = average number of ships present

e = Napierian logarithmic base, 2.71828

The Poisson equation makes it possible to calculate the distribution of ship arrivals if only the average number of ships present during the period in question is known. For example, if it is known that an average number of ships present at a certain port is 8.0, the number of hours during a one-year period (8760 hr) that 10 ships will be present is

$$F_{10} = \frac{8760(8)^{10}e^{-8}}{10!} = 872 \ ?$$

869

This and similar values are plotted in Fig. 21-6, which shows the Poisson distribution for $\bar{n} = 8$ and a period of one year. The calculations indicate that at a 9-berth port there are 872 hr during the year in which one ship will be waiting for a berth. Similarly, at a port with 11 berths, there will be 872 hr during the year in which 1 berth will be vacant.

Since there are costs involved in idle berths as well as in waiting ships, the optimum number of berths is a compromise, best determined by minimization of the sum of the annual costs of vacant berths and waiting ships.[1] Using this approach, Nicolaou [5] has developed a useful graph, shown in Fig. 21-7, that relates annual port capacity to the number of berths.

It will be noted that two parameters are shown in Fig. 21-7, percent of congestion and percent of occupancy. Percent of congestion is defined as the percent of time for which the number of ships in port exceeds the number of berths available. Percent of occupancy is defined as the percent of time that the total number of berths available at the port, N_B, are occupied.

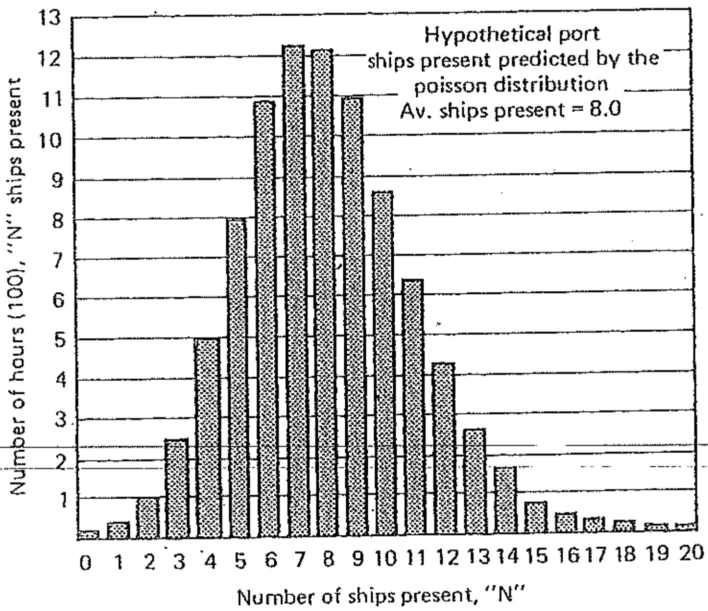
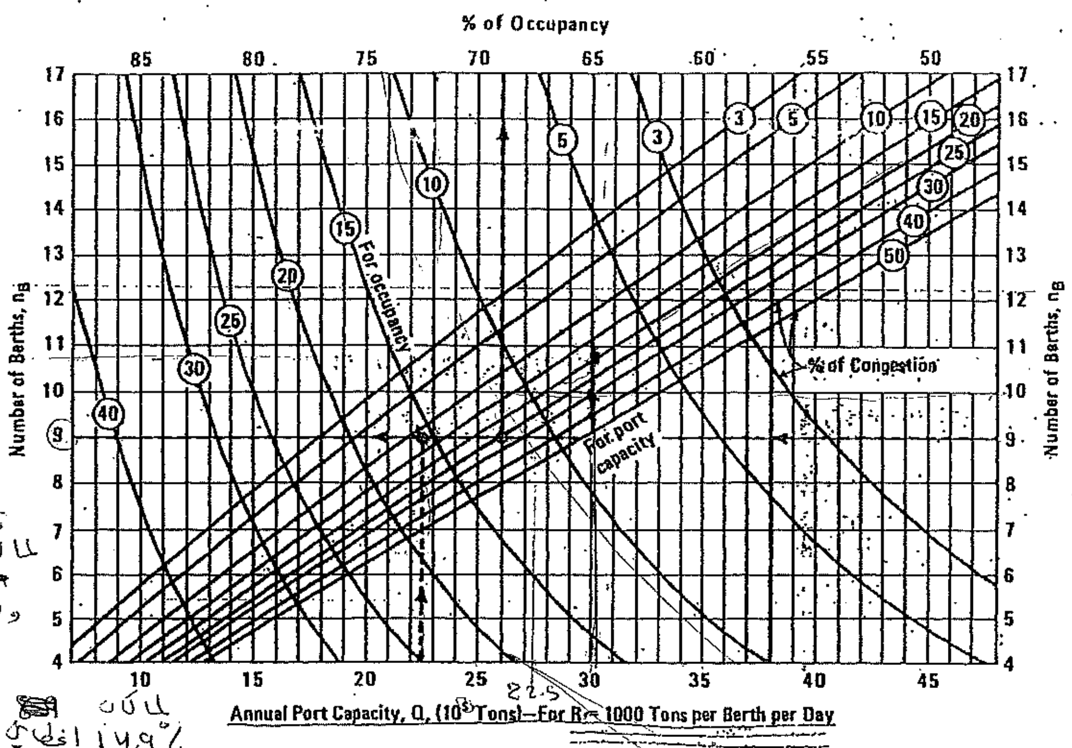


Figure 21-6 Ship distribution at a hypothetical port. (Courtesy American Society of Civil Engineers.)

[1]Wanhill [7] has described an iterative procedure based on a wider criterion: The minimization of the total port usage cost including the service time costs as well as the costs of the ship's waiting time and the costs of providing the berths.



8.7 N_B 14.9%

9 N_B ⇒ ≈ 12%

Figure 21-7 Relationship between annual port capacity, congestion, and berth occupancy. (Source: S. N. Nicolaou, "Berth Planning by Evaluation of Congestion and Cost," *Proceedings*, American Society of Civil Engineers, November 1967.)

Nicolaou [5] showed that percent of congestion is related to average cost of a ship waiting for a berth, C_S, and the average cost of an idle berth, C_B, by the following inequality:

$$\text{Percent congestion} < 100\left(1 - \frac{C_S}{C_B + C_S}\right) \tag{21-3}$$

The annual port capacity, Q, is given by the following equation:

$$Q = N_B R T\left(\frac{\text{percent occupancy}}{100}\right) \tag{21-4}$$

where

N_B = number of berths

R = annual average cargo-handling rate per berth, tons/day

T = time period, usually 365 days

The use of Fig. 21-7 is illustrated by a numerical example.

EXAMPLE 21-1

Given: $T = 365$ days; $Q = 1.8 \times 10^6$ tons; $R = 800$ tons/day; $C_B = \$350$; and $C_S = \$2000$. Then let

$$Q' = Q\left(\frac{1000}{R}\right) = (1.8 \times 10^6)\left(\frac{1000}{800}\right) = \boxed{2.25 \times 10^6 \text{ tons}}$$

By Eq. 21-3,

$$\text{Percent congestion} < 100\left(1 - \frac{2000}{350 + 2000}\right)$$

$$\text{Percent congestion} < 14.9\%$$

By entering Fig. 21-7 with a value of $Q' = 2.25 \times 10^6$ tons and for a value of percent congestion less than 14.9 (interpolated from the straight-sloped lines), we find that the chart gives a value $N_B = 9$. Note that $N_B = 8$ gives a percent congestion of 24, which is too large. Corresponding to $N_B = 9$, the percent congestion is 12 percent. Reentering the chart at the right ordinate with a value of $N_B = 9$ and intersecting the percent of congestion curves as shown, we find that

$$\text{Percent occupancy} = 69.0$$

from which

$$Q = 9(800)(365)\left(\frac{69.0}{100}\right) = 1.813 \times 10^6 \text{ tons}$$

Figure 21-7 should give suitable planning estimates of the number of required berths for the range of values shown. More complete analyses, which are beyond the scope of this text, may be made by the use of queuing theory [8] or by computer simulation [9, 10].

21-7. BERTH AND SLIP DIMENSIONS

The space required alongside a wharf for the berthing depends on:

1. Size and type of ships served
2. Wharf configuration
3. Mooring procedures

The required berth length is equal to the length of a ship plus a small clearance between adjacent ships and space for the ship's lines. Thus a typical berth length for noncontainer general cargo ships is about 230 to 260 m (750 to 850 ft). Container ships require a berth length of about 305 m (1000 ft). Dry bulk terminals commonly have a berth length of about 260 m (850 ft), whereas berth lengths at bulk petroleum terminals vary from approximately 245 to 265 m (800 to 1200 ft), depending on the length of the largest ship served.

For a wharf configuration utilizing two-berth piers, the required slip width is roughly the sum of the beams of two ships and the length of a tugboat. When four-berth piers are used, minimum slip width is based on the beams of two ships (moored) plus the beam of another ship (mooring) plus the length of a tugboat.

21-8. BREAK-BULK GENERAL CARGO TERMINALS

At break-bulk general cargo terminals, the cargo is transferred between the ship and the wharf using various types of equipment, including mobile cranes on the wharf, gantry cranes, and the booms and winches on the ship (ship's gear). The most common method found at U.S. ports is the use of ship's gear operated by a gang of about 15 longshoremen [2]. The cargo is usually moved between the working area of the wharf and the storage area by trains of small cargo cars pulled by a tractor or by forklift trucks.

From U.S. ports, break-bulk cargo is most commonly transported inland by truck, although about 25 percent goes by rail.

At a typical break-bulk terminal, temporary dry storage is provided in a large building called a transit shed. In addition to providing short-term storage, the transit shed may also provide space for customs activities and for port administration and security. The transit shed should not be used for long-term storage, and when long-term storage facilities are required, warehouses and open storage areas usually are located shoreward of the transit shed.

Experience has shown that for a typical dry-cargo ship, a transit shed that has an area of 7900 to 9300 m^2 (85,000 to 100,000 ft^2) is desired. This value provides adequate space to accommodate a single average-sized, dry-cargo vessel and includes an allowance of about 40 percent for aisles and other nonstorage areas. With larger vessels currently in service, transit sheds as large as 11,000 m^2 (120,000 ft^2) have been provided at some major ports [11]. Proportionately smaller transit sheds may be used at small terminals where ships are expected to discharge and take on only partial loads.

A common length of transit shed is 150 to 165 m (500 to 550 ft). The larger value provides a sheltered storage area adjacent to practically the entire length of a large merchant cargo vessel. A minimum transit shed width of 50 m (165 ft) is recommended for marginal wharf terminals. (See, e.g., Fig. 21-8). Where a single shed serves two berths, one on each side of a pier, approximately twice this value would be required to provide the needed area per berth. The choice of configuration of the transit shed may take many forms and will be governed by local conditions [11].

Figure 21-8 Port facilities at Port Everglades, Florida. (Courtesy The American Association of Port Authorities.)

Table 21-5. Principal Dimensions for General Cargo Terminals

Berth lengths	
Wharves	229 m (750 ft) multiples
Piers	259 m (850 ft)
Apron widths	
No railroad tracks	9 m (30 ft)
One railroad track	9 m (30 ft)
Two railroad tracks	11.7 m (38.5 ft)
Clear stacking height: sheds	6.1 m (20 ft)
Space per berth: gross transit shed	4650–11,150 m^2 (50,000–120,000 ft^2)
Interior column spacing	12.2 m (40 ft) minimum

Source: Port Planning, Design and Construction, American Association of Port Authorities, Inc., Washington, DC, November 1973.

A 6-m- (20-ft-) wide covered platform along the rear (shoreward) side of the transit shed is recommended to facilitate the transfer of cargo between trucks and railroad cars and the transit shed. Door openings usually are provided in alternate bays along both the wharf side and the platform side of the transit shed.

The portion of the wharf or pier that lies between the waterfront and the transit shed is called the apron. This uncovered space is needed for mooring and for the loading, unloading, and movement of cargo into the transit shed. Along the waterfront edge of the apron, space must be reserved for bollards, cleats, and other mooring devices. Connections for electric power and for telephone and water service must be provided. These connections usually are housed in service boxes built into the deck of the apron along the waterfront edge. At least two services boxes per berth are recommended.

When railroad service is desired along the apron, the rails are constructed flush with the apron deck, and the tracks are constructed on 4.1-m (13.5-ft) center-to-center spacing. Rail-supported gantry cranes may be installed along the apron, in which case 1 to 1.5 m (3 to 5 ft) of apron width must be allocated for each crane rail.

The total apron width will vary from about 6 m (20 ft) to more than 18 m (60 ft) depending on the facilities provided. Table 21-5 gives principal dimensions for general cargo terminals recommended by the American Association of Port Authorities [11]. Figure 21-9 shows a cross section of a typical transit shed and apron.

21-9. NEO-BULK GENERAL CARGO TERMINALS

Neo-bulk general cargo differs from break-bulk general cargo mainly in that the packages are larger. At neo-bulk terminals, ship–apron transfer may be done by ship's gear, by shoreside cranes, or in the case of automobiles, by longshoremen driving single vehicles. Except for automobiles, cargo is usually transferred between the apron and storage area by forklifts. Transport to and from inland points is by highway truck and less commonly by rail car.

A neo-bulk general cargo terminal normally does not have a transit shed. Rather storage and classification of the cargo occurs in an open backland area of approximately 14 hectares (35 acres) for a single berth [2]. A sketch showing typical elements of a neo-bulk general cargo terminal is shown in Fig. 21-10.

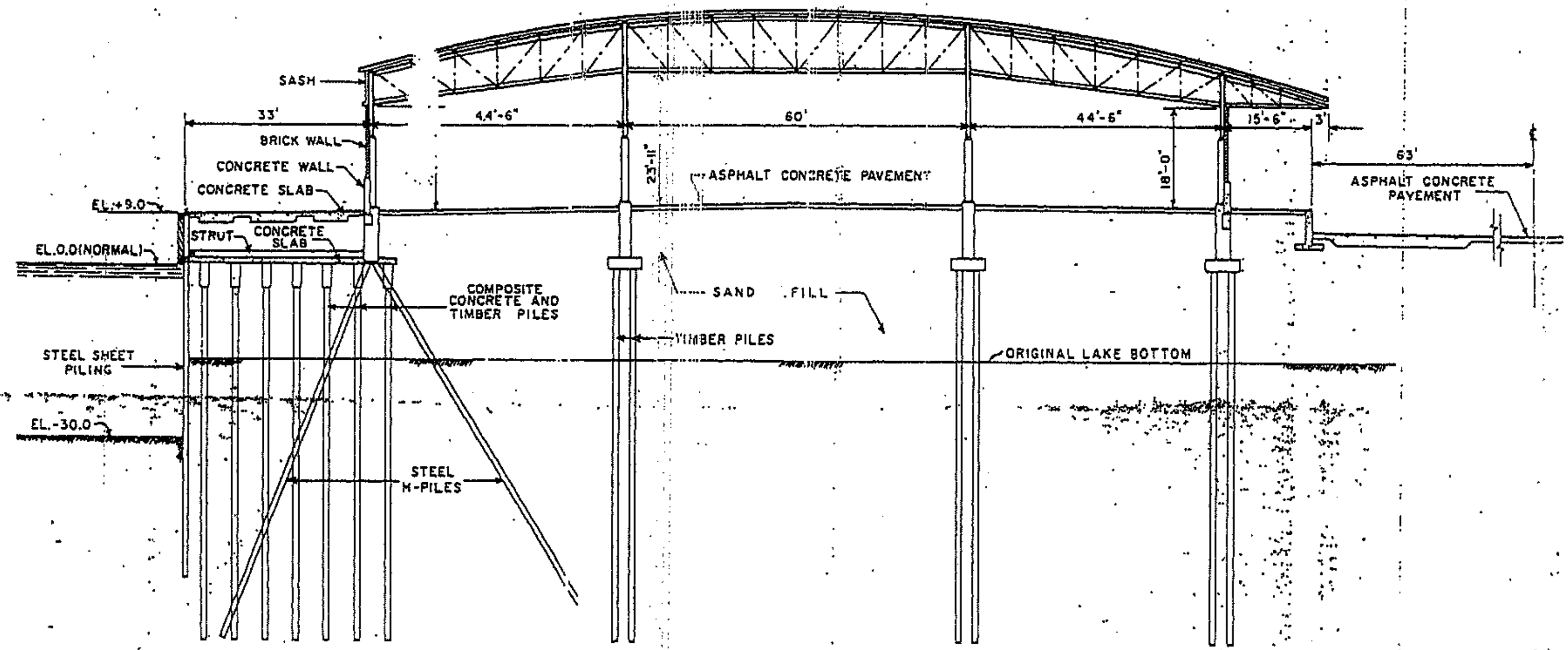


Figure 21-9 Typical cross section, South Pier No. 2, Milwaukee, Wisconsin. (Courtesy The American Association of Port Authorities.)

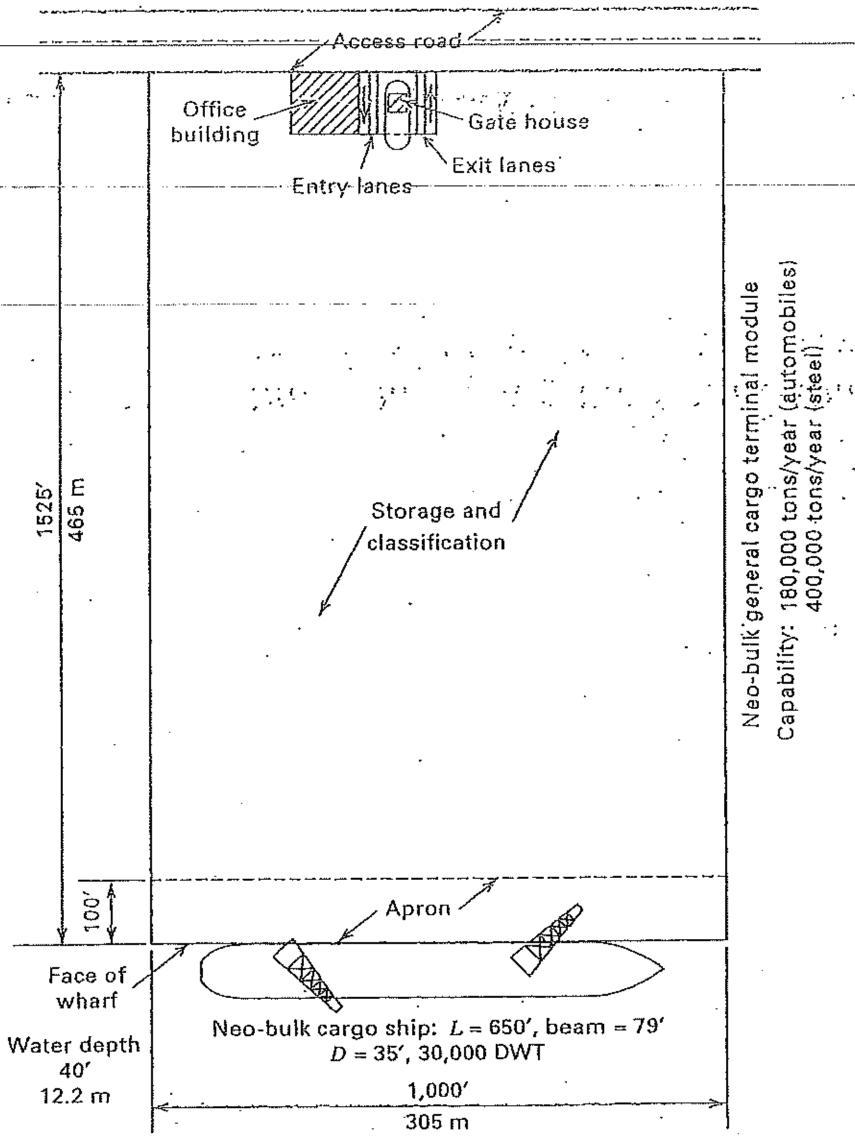


Figure 21-10 Elements of a typical neo-bulk general cargo terminal. (Source: Hockney, Lester A. and Lawrence L. Whiteneck, *Port Handbook for Estimating Marine Terminal Cargo Handling and Capability*, Maritime Administration, Washington DC, November 1986.)

21-10. CONTAINER PORTS

Prior to 1956, cargo at ocean ports was handled in small units. This package-by-package cargo-handling procedure was time-consuming and costly, accounting for a large percentage of the overall costs of ocean shipping.

In 1956 and 1957, Sea-Land Service, Inc. and Matson Navigation Company independently and almost simultaneously introduced a new concept in ocean shipping that came to be known as containerization [12]. It has been said that containerization has made perhaps the greatest impact on the shipping industry since the invention of the steam engine.

Containers are simply boxes, typically 6.06 or 12.19 m (20 or 40 ft) in length and an outside width and height of 2.44 m (8 ft). Table 21-6 gives the standard dimensions and weights of containers proposed by the International Standardization Organization. Various sizes of containers have been used, and some shipping companies have recently been experimenting with nonstandard containers with larger dimensions and heavier maximum weights [13].

Table 21-6 Standard Dimensions and Weights of Containers

Length	Weight	
	Empty	Maximum Loaded
6.06 m (20 ft)	19–22 kN (4270–4945 lb)	240 kN (53,954 lb)
12.19 m (40 ft)	28–36 kN (6295–8903 lb)	305 kN (68,567 lb)

Source: Brunn, Per, *Port Engineering,* 4th ed., Gulf Publishing Company, Houston, TX, 1989.

The usage of containers eliminates much of the handling of small units of cargo at a port facility. A container usually is loaded and sealed at the place of origin and remains sealed until it arrives at its destination. However, a fraction (up to 40 percent) of the containers are usually loaded (stuffed) and unloaded (stripped) at the port.

Containers may arrive at the port facility by rail but more commonly arrive by trucks. Upon arrival the containers are weighed, logged in, and stored temporarily in an assigned location in a marshaling area. The containers are later moved from the marshaling area to wharfside and transferred to a ship by means of heavy container cranes. This procedure is reversed at the destination end of the trip. The principal elements of a large container port may be seen in Fig. 21-11.

At least five advantages may be listed for a container port:

1. The berth capacity is great, often being five times as high as the capacity of a traditional general cargo berth.
2. Overall transit time is less.
3. Container ports offer greater safety for waterfront employees.
4. There is less damage to cargo.
5. Less pilferage occurs at a container port.

The principal disadvantage of a container port is the large amount of land required for the marshaling area. The need to provide large expensive container cranes is also a disadvantage of container ports.

Planning considerations and design criteria for container ports, largely abstracted from reference 11, are described briefly in the following paragraphs.

Since containers are designed to be interchangeable between ship, truck, and rail transportation, it is important that container ports be easily accessible by highways and railroads. Preferably, container berths should be placed along a marginal wharf rather than a pier because of the need to provide a large supporting area for each berth.

The components that comprise a typical container terminal are (1) the ship berth, (2) the container cranes, (3) the marshaling area, (4) the container packing shed, (5) the entry facilities, and (6) the garage and inspection building. Most of these components can be seen in Fig. 21-11.

Ship Berths. The berth lengths will depend upon the size of the vessel to be accommodated. The largest containerships in service have a length of about 290 m (950 ft), a width of about 34 m (110 ft), and a draft of approximately 10.6 m (35 ft). The American Association of Port Authorities [11] recommends a minimum berth length of 259 m (850 ft) up to a maximum of 305 m (1000 ft) to accommodate the largest containership in service. If the container berths are arranged along a marginal wharf, a certain amount of flexibility in berth length will be provided, and the choice of a design berth length is not critical.

Cranes. Containerships usually do not have shipboard cranes, and container cranes on shore must be provided. Normally, two or more cranes working simultaneously will load

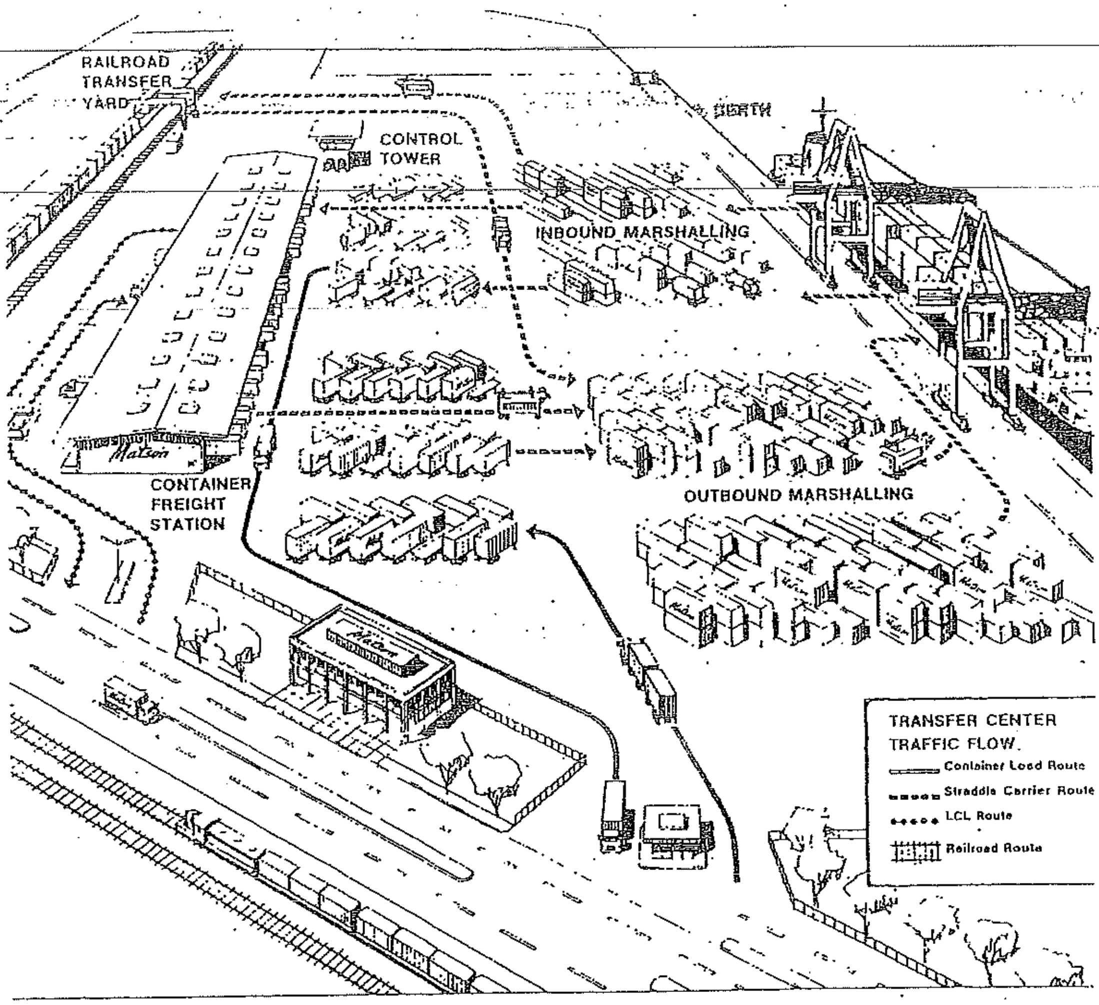


Figure 21-11 Example of a containership terminal and traffic flow. The drawing illustrates how the flow of containers is organized in a Matson transfer center. The arrows show how less-than-container-load cargo moves to the container freight station first and comes out of the container freight station last. Full container shipments go direct to the container yard. The rest of the movement within the transfer center is handled by straddle trucks and gantry cranes. (Courtesy The American Association of Port Authorities.)

and unload a containership. These cranes are rail mounted on the dock and typically have a capacity of 20 to 30 metric tons (20 to 30 long tons.) (See Fig. 21-12.)

Marshaling Areas. Two general classes of container storage are used for the marshaling area: chassis storage and stacked storage. With chassis storage, a container unloaded from a ship is placed on a semitrailer chassis and then hauled by a yard tractor (Fig. 21-13*a*) to an assigned space in the terminal where it remains until retrieved by a highway tractor. Similarly, arriving export containers are assigned locations in the marshaling yard and remain there until being moved to the loading area by yard tractors.

The chassis storage system provides operational efficiency, as each container is immediately available to a tractor unit. However, chassis storage requires more yard space and more chassis than do other systems.

In chassis storage areas containers are commonly stored back to back with at least 1.2 m (4 ft) between the backs of containers. The aisles between the rows should be at least 20 m (65 ft) in width and preferably 21 to 23 m (70 to 75 ft) to facilitate the parking

Figure 21-12 A containership terminal. (Courtesy The Port of Long Beach.)

maneuvers. The parking spaces are typically 3 m (10 ft) wide, and the length is determined by the size of the container to be parked [14].

In the stacked storage system containers are stacked by straddle carriers or by a traveling bridge crane. Straddle carriers are special pieces of equipment that can straddle the containers and transport them between shipside and storage areas or onto trucks or railroad cars (see Fig. 21-13*b*). Straddle carriers can stack containers two or three high in long rows that are one container in width. In these yards it is usually advantageous to orient the container at a 45-degree angle to the traffic flow patterns. Traveling bridge cranes stack the containers three or four high. (See Fig. 21-13*c*). These yards usually are laid out at right angles to the traffic flow [14]. Stacked storage systems require considerably less area than do chassis storage systems but are not as efficient because nonproductive handlings are required to retrieve containers.

Container Packing Shed. A container packing shed normally is provided where less than container load (LTCL) shipments are handled. It need not be contiguous to the marshaling area and definitely should not be placed at the normal location of a transit shed. There it would tend to encumber the steady movement of containers to and from the cranes during loading and unloading operations. The size of the packing shed varies widely. Its general configuration resembles a typical truck terminal, allowing delivery trucks to arrive at one side of the building and the cargo to be moved from these trucks directly into waiting containers on the opposite side. Thus, the building tends to be long and narrow with emphasis on the number of truck and container doors necessary. As an example, one of the major packing sheds at the Elizabeth, New Jersey, container port is 305 m (1000 ft) long and 30.5 m (100 ft) wide with a total of 162 truck doors [11].

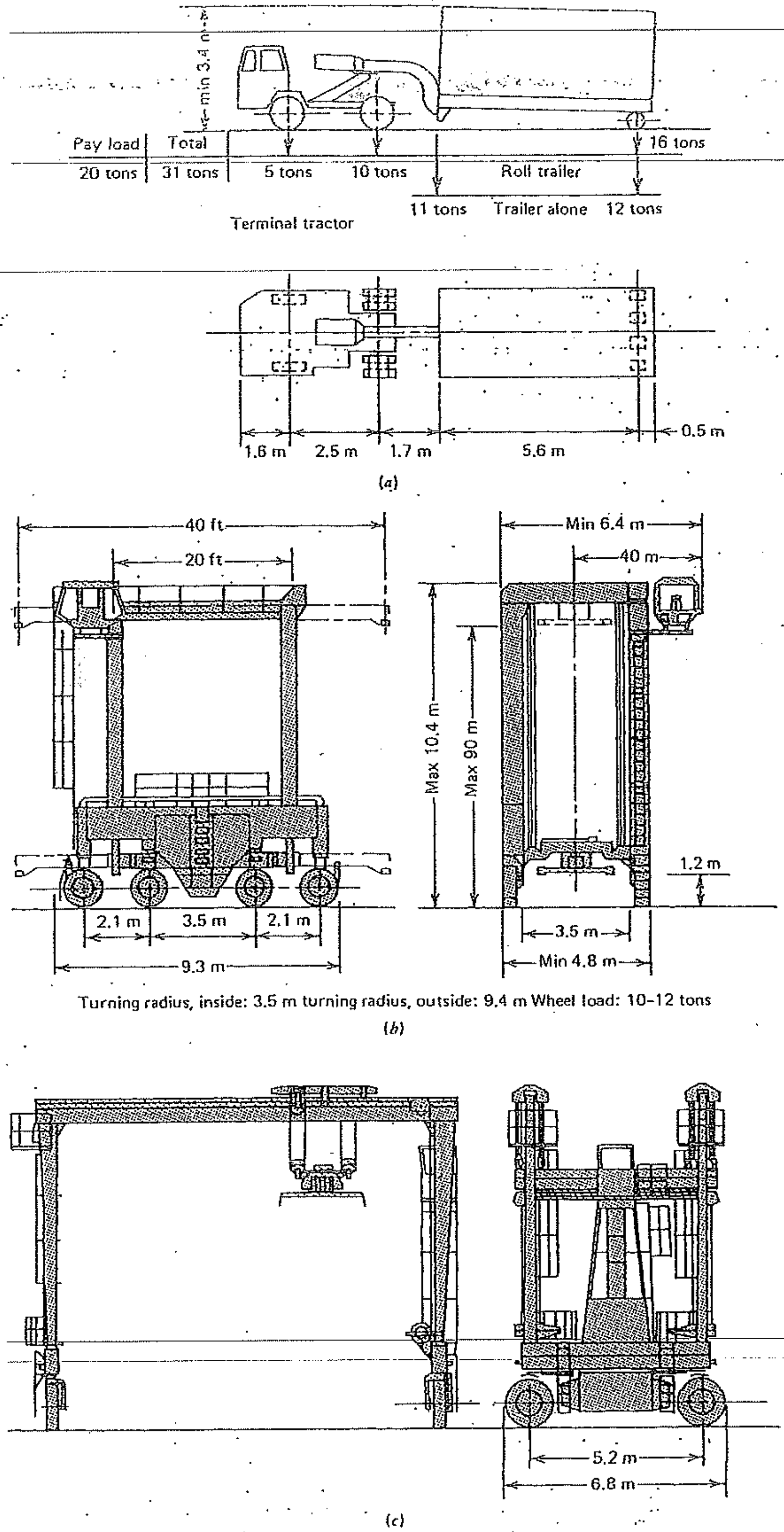


Figure 21-13 Equipment used to transport containers at a container terminal. (*a*) Typical yard tractor and 20-ft trailer. (*b*) Typical straddle carrier. (*c*) Typical gantry crane on rubber wheels. (Source: *Planning and Design of Ports and Marine Terminals*, by Agerschou, Hans, et al., John Wiley & Sons, New York, 1983.)

Entry Facilities. The truck entrance to the terminal facility usually consists of two to six lanes in each direction. Each lane is normally provided with a truck scale to weigh the entering and departing containers. A receiving and delivery office usually is located at the entry/exit point to handle the necessary paperwork and to assign storage positions to incoming containers.

Container ports need to be served by major traffic arteries. Container operations generate substantial truck traffic, especially on days that ships are in port. This peaking necessitates truck queues waiting to enter the terminal, and unless generous approach roads are provided, enormous congestion will result.

Garage and Inspection Building. A small building for the physical inspection of arriving or departing containers normally is located in the vicinity of the entrance and adjacent to the marshaling area. In addition, a garage may be provided to maintain the stevedoring devices used to handle the containers in the marshaling yard.

Direct transfer of containers from rail to ship is not generally feasible. In such a transfer, the string of rail cars would have to be moved continuously during the loading and unloading operation to position the containers under the crane. Instead, it is usually better to unload the containers off-site and transport them to the marshaling area on rubber tires. Therefore, tracks will not be required at the stringpiece, although they should be provided to the packing shed.

21-11. ROLL-ON/ROLL-OFF FACILITIES

Because of the high investment costs of container ports with conventional lift-on/lift-off (LO/LO) facilities, many ports have developed facilities for roll-on/roll-off (RO/RO) cargo handling. These systems are often combined with conventional handling systems.

The RO/RO service can be provided by three basic methods [13]:

1. The tractor and trailer unit drives on, remains on the ship during its voyage, and drives off at the destination port. This procedure is generally suitable only for short trips since the tractor unit utilizes space and remains idle during the sea journey.
2. The tractor unit tows the container from the storage area onto the ship and drives off, leaving only the trailer and container for the sea journey. Another tractor will be required to tow the trailer off the ship at the destination port. The tractor unit may be a normal highway unit or a smaller unit used only in the port area.
3. A tractor unit tows the trailer and container onto the ship where a ship crane lifts it off and stores it, or else a straddle carrier transports the container aboard and stacks it on the ship. Only the container remains on the ship during the sea trip. The process is reversed at the destination port.

The RO/RO operations may also vary because of differences in ship design. Loading ramps may be provided in the bow, stern, or side of the ship.

Where RO/RO operations are used, special mooring devices may be required to hold the ship properly in place. Special adjustable ramps may be required on the wharf to accommodate variation in ship design and in the tide. Where cargo is handled through the side of a vessel, additional dock apron area will be needed to allow for ramp clearance. Since RO/RO loadings and unloadings can be performed in a short period of time, a large service area will be required for the processing and temporary storage of the freight.

21-12. LIGHTER-ABOARD-SHIP TERMINALS

A novel unitized cargo concept designed to maximize utilization of oceangoing ships and terminal facilities is the lighter-aboard-ship system. With this system, cargo is loaded onto

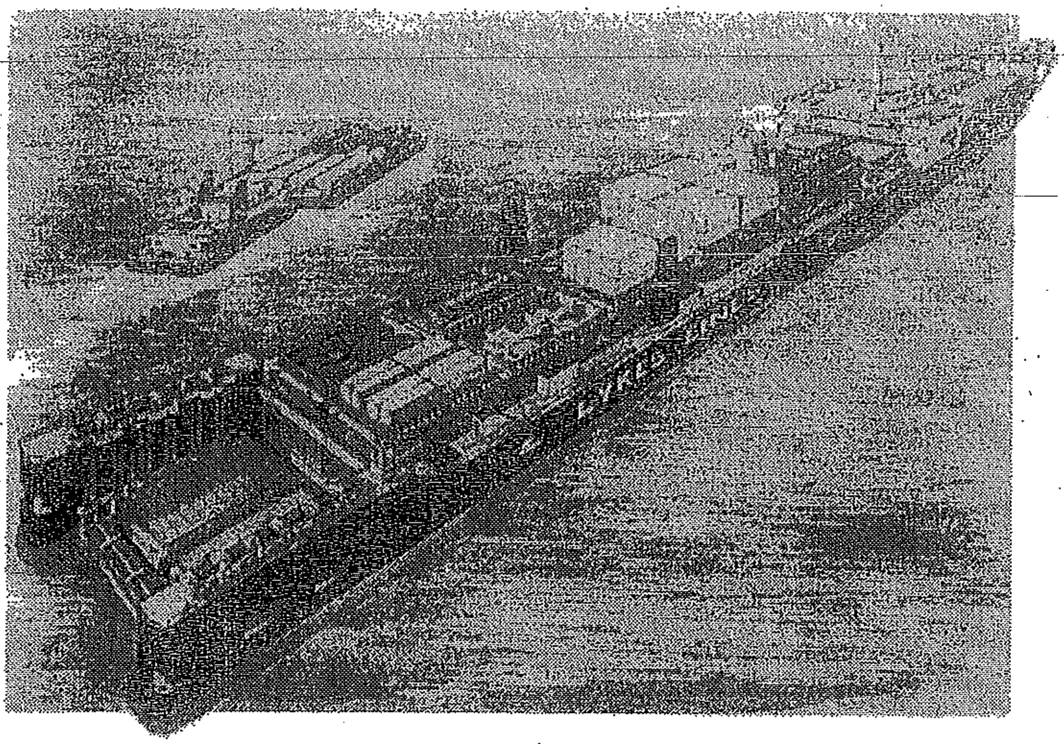


Figure 21-14 A seabee vessel. (Source: Lykes Lines.)

large barges (lighters), then the barges and cargo are placed aboard a specially designed mother ship for the sea journey. The lighters either are loaded aboard the mother ship by a large crane on an overhang at the stern or are lifted by an elevator onto the ship after being floated into a stern well. In the destination harbor the barges and cargo are unloaded and moved to a port facility where the cargo is unloaded from the barges (See Fig. 21-14).

The mother ship may not need a land berth; however, some of the lighter-carrying ships are designed to transport both lighters and containers and therefore must have access to a container port.

Special requirements of lighter-aboard-ship terminals include facilities for lighter berthing and loading, berthing areas for tugboats, and a storage basin for the empty and loaded lighters.

More information on the functional planning of lighter-aboard-ship terminals is given in reference 11.

21-13. DRY BULK CARGO TERMINALS

A wide variety of materials can best be transported unpackaged without the use of containers or pallets. Such commodities, termed dry bulk cargo, include grain, coal, sand, gravel, salt, sugar, scrap metal, and ores. Specialized port facilities are required to handle dry bulk cargo. These facilities may be placed on-shore or off-shore but preferably are separated from general cargo and container ports.

Dry bulk cargo ships tend to be larger than general cargo ships and may require deeper channels and berths, more rugged fender systems, and more extensive storage areas. The storage areas may be open, enclosed, or a combination of both, depending on the nature of the material to be handled at the port.

Cargo-handling systems are varied, depending on the characteristics of the materials being handled. They include belt conveyor systems, cranes with clam shell buckets,

Table 12-8A Stopping Sight Distance on Wet Pavements

Design Speed (mph)	Assumed Speed for Condition (mph)	Brake Reaction Time (sec)	Brake Reaction Distance (ft)	Coefficient of Friction, f	Braking Distance on Level[a] (ft)	Stopping Sight Distance Computed[a] (ft)	Stopping Sight Distance Rounded for Design (ft)
20	20–20	2.5	73.3–73.3	0.40	33.3–33.3	106.7–106.7	125–125
25	24–25	2.5	88.0–91.7	0.38	50.5–54.8	138.5–146.5	150–150
30	28–30	2.5	102.7–110.0	0.35	74.7–85.7	177.3–195.7	200–200
35	32–35	2.5	117.3–128.3	0.34	100.4–120.1	217.7–248.4	225–250
40	36–40	2.5	132.0–146.7	0.32	135.0–166.7	267.0–313.3	275–325
45	40–45	2.5	146.7–165.0	0.31	172.0–217.7	318.7–382.7	325–400
50	44–50	2.5	161.3–183.3	0.30	215.1–277.8	376.4–461.1	400–475
55	48–55	2.5	176.0–201.7	0.30	256.0–336.1	432.0–537.8	450–550
60	52–60	2.5	190.7–220.0	0.29	310.8–413.8	501.5–633.8	525–650
65	55–65	2.5	201.7–238.3	0.29	347.7–485.6	549.4–724.0	550–725
70	58–70	2.5	212.7–256.7	0.28	400.5–583.3	613.1–840.0	625–850

[a]Different values for the same speed result from using unequal coefficients of friction.

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Table 12-9A Design Controls for Crest Vertical Curves Based on Stopping Sight Distance

Design Speed (mph)	Assumed Speed for Condition (mph)	Coefficient of Friction, f	Stopping Sight Distance Rounded for Design (ft)	Rate of Vertical Curvature, K [length (ft) per percent of A] Computed[a]	Rate of Vertical Curvature, K Rounded for Design
20	20–20	0.40	125–125	8.6–8.6	10–10
25	24–25	0.38	150–150	14.4–16.1	20–20
30	28–30	0.35	200–200	23.7–28.8	30–30
35	32–35	0.34	225–250	35.7–46.4	40–50
40	36–40	0.32	275–325	53.6–73.9	60–80
45	40–45	0.31	325–400	76.4–110.2	80–120
50	44–50	0.30	400–475	106.6–160.0	110–160
55	48–55	0.30	450–550	140.4–217.6	150–220
60	52–60	0.29	525–650	189.2–302.3	190–310
65	55–65	0.29	550–725	227.1–394.3	230–400
70	58–70	0.28	625–850	282.8–530.9	290–540

[a]Using computed values of stopping sight distance.

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Table 12-10A Design Controls for Sag Vertical Curves Based on Stopping Sight Distance

Design Speed (mph)	Assumed Speed for Condition (mph)	Coefficient of Friction, f	Stopping Sight Distance Rounded for Design (ft)	Rate of Vertical Curvature, K [length (ft) per percent of A] Computed[a]	Rounded for Design
20	20–20	0.40	125–125	14.7–14.7	20–20
25	24–25	0.38	150–150	21.7–23.5	30–30
30	28–30	0.35	200–200	30.8–35.3	40–50
35	32–35	0.34	225–250	40.8–48.6	50–50
40	36–40	0.32	275–325	53.4–65.6	60–70
45	40–45	0.31	325–400	67.0–84.2	70–90
50	44–50	0.30	400–475	82.5–105.6	90–110
55	48–55	0.30	450–550	97.6–126.7	100–130
60	52–60	0.29	525–650	116.7–153.4	120–160
65	55–65	0.29	550–725	129.9–178.6	130–180
70	58–70	0.28	625–850	147.7–211.3	150–220

[a]Using computed values of stopping sight distance.

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Table 12-11A Design Controls for Crest Vertical Curves Based on Passing Sight Distance

Design Speed (mph)	Minimum Passing Sight Distance, Rounded for Design (ft)	Rate of Vertical Curvature, K,[a] Rounded for Design [length (ft) per percent of A]
20	800	210
25	950	300
30	1100	400
35	1300	550
40	1500	730
45	1650	890
50	1800	1050
55	1950	1230
60	2100	1430
65	2300	1720
70	2500	2030

[a]Computed from rounded values of passing sight distance.

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Table 13-2A Clear-Zone Distances in Feet from Edge of Driving Lane

Design Speed (mph)	Design ADT	Distance for Fill Slopes			Distance for Cut Slopes		
		6:1 or Flatter	5:1 to 4:1	3:1[a]	3:1	5:1 to 4:1	6:1 or Flatter
40 or less	Under 750	7–10	7–10		7–10	7–10	7–10
	750–1500	10–12	12–14		10–12	10–12	10–12
	1500–6000	12–14	14–16		12–14	12–14	12–14
	Over 6000	14–16	16–18		14–16	14–16	14–16
45–50	Under 750	10–12	12–14		8–10	8–10	10–12
	750–1500	12–14	16–20		10–12	12–14	14–16
	1500–6000	16–18	20–26		12–14	14–16	16–18
	Over 6000	18–20	24–28		14–16	18–20	20–22
55	Under 750	12–14	14–18		8–10	10–12	10–12
	750–1500	16–18	20–24		10–12	14–16	16–18
	1500–6000	20–22	24–30		14–16	16–18	20–22
	Over 6000	22–24	26–32[b]		16–18	20–22	22–24
60	Under 750	16–18	20–24		10–12	12–14	14–16
	750–1500	20–24	26–32[b]		12–14	16–18	20–22
	1500–6000	26–30	32–40[b]		14–18	18–22	24–26
	Over 6000	30–32[b]	36–44[b]		20–22	24–26	26–28
65–70	Under 750	18–20	20–26		10–12	14–16	14–16
	750–1500	24–26	28–36[b]		12–16	18–20	20–22
	1500–6000	28–32[b]	34–42[b]		16–20	22–24	26–28
	Over 6000	30–34[b]	38–46[b]		22–24	26–30	28–30

[a]Since recovery is less likely on the unshielded, traversable 3:1 slopes, fixed objects should not be present in the vicinity of the toe of these slopes. Recovery of high-speed vehicles that encroach beyond the edge of the shoulder may be expected to occur beyond the toe of slope. Determination of the width of the recovery area at the toe of the slope should take into consideration right-of-way availability, environmental concerns, economic factors, safety needs, and accident histories. Also, the distance between the edge of the travel lane and the beginning of the 3:1 slope should influence the recovery area provided at the toe of the slope.

[b]Where a site-specific investigation indicates a high probability of continuing accidents or such occurrences are indicated by accident history, the designer may provide clear-zone distances greater than 30 ft as indicated. Clear zones may be limited to 30 ft for practicality and to provide a consistent roadway template if previous experience with similar projects or designs indicates satisfactory performance.

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Table 13-6A Minimum Edge of Pavement Designs for Turns at Intersections: Simple Curves and Tapers

Angle of Turn (deg)	Design Vehicle	Simple Curve Radius	Simple Curve Radius with Taper: Radius (ft)	Offset (ft)	Taper (ft:ft)
75	P	35	25	2.0	10:1
	SU	55	45	2.0	10:1
	WB-40	—	60	2.0	15:1
	WB-50	—	65	3.0	15:1
	WB-62	—	140	4.0	20:1
90	P	30	20	2.5	10:1
	SU	50	40	2.0	10:1
	WB-40	—	45	4.0	10:1
	WB-50	—	60	4.0	15:1
	WB-62	—	120	4.0	30:1
105	P	—	20	2.5	8:1
	SU	—	35	3.0	10:1
	WB-40	—	40	4.0	10:1
	WB-50	—	55	4.0	15:1
	WB-62	—	115	3.0	30:1

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Table 13-7A Minimum Edge of Pavement Designs for Turns at Intersections: Three-Centered Compound Curves

Angle of Turn (deg)	Design Vehicle	Curve Radii (ft)	Symmetric Offset (ft)	Curve Radii (ft)	Asymmetric Offset (ft)
75	P	100–25–100	2.0	—	—
	SU	120–45–120	2.0	—	—
	WB-40	120–45–120	5.0	120–45–200	2.0–6.5
	WB-50	150–50–150	6.0	150–50–225	2.0–10.0
	WB-62	440–75–440	15.0	140–100–540	5.0–12.0
90	P	100–20–100	2.5	—	—
	SU	120–40–120	2.0	—	—
	WB-40	120–40–120	5.0	120–40–200	2.0–6.0
	WB-50	180–60–180	6.0	120–40–200	2.0–10.0
	WB-62	400–70–400	10.0	160–70–360	6.0–10.0
105	P	100–20–100	2.5	—	—
	SU	100–35–100	3.0	—	—
	WB-40	100–35–100	5.0	100–55–100	2.0–8.0
	WB-50	180–45–180	8.0	150–40–210	2.0–10.0
	WB-62	520–50–520	15.0	360–75–600	4.0–10.5

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1 km = 0.621371192 mile

1.852 Km/hr = 1 knots