CORRUGATED STEEL PIPE
Design Manual

CSP detention systems can be made to fit any site configuration.

Corrugated steel structural plate bridge structure.

CHAPTER one

INTRODUCTION

Corrugated steel pipe continues to play a major role in modern engineering technology for drainage systems. The versatility of corrugated steel pipe allows a designer flexibility to match the right product with the application, whether drainage or non-drainage. This chapter will briefly describe just a few of the many ways in which this product is commonly utilized. Technical information on applications is more completely defined in the later chapters.

100+ Years of History and Service

Invented in 1896, corrugated steel pipe (CSP) has been successfully used for over 100 years. Millions of installations have shown CSP to be a product at the forefront of modern engineering technology, a proven material of choice in both drainage and non-drainage applications.

Versatility

Corrugated steel pipe is manufactured in a wide range of shapes and sizes, including diameters from 6 inches to over 50 feet, as well as arch and box designs in a multitude of span and rise dimension combinations, with spans exceeding 80 feet.

Fabrication in many configurations and designs allows CSP great versatility; lending itself to a virtually endless array of fittings for drainage systems such as wyes, tees, elbows, manholes, sediment pond risers and detention systems. Corrugated steel pipe is the choice of engineers in the development of innovative design.

Structural Strength

The mechanical properties of steel are controlled at the mill, and the finished product is fabricated to exacting specifications. The strength and integrity of steel-soil interaction structures is almost unlimited. CSP structures can handle fill heights in excess of 100 feet and the product's beam strength allows structures to be manufactured in lengths exceeding any alternative drainage product on the market.

Durability

Corrugated steel pipe is available with a wide variety of protective coatings that have proven to meet the requirements of demanding environments. No matter the location or application, CSP has a coating to meet the needs of the situation. This provides the engineer or contractor an end result of optimum service life for the structure at the lowest cost. Service life exceeding 100 years can be obtained using the proper coating, specific to location and application.

Economic Value

A cost-effective product that is easily and quickly installed, and one which can be manufactured in longer lengths, CSP provides a viable alternative to other materials. These efficiency attributes help lessen the amount of equipment and time required to complete the job. Corrugated steel pipe fabricating plants are located throughout the United States, resulting in quick delivery of the product. CSP is not sensitive to temperature or moisture extremes and can be installed even during inclement weather. Rapid installation and the inherent strength of steel enable the contractor to make more efficient use of equipment. Heavy earthmovers can operate over corrugated steel structures after a proper covering with soil, shortening the time trenches must be left open and allowing the project to progress quickly. Corrugated steel pipe is the choice of cost conscious contractors and project owners, combining a strong durable product with quick installation.

Recycled Content

Steel contains the highest percentage of recycled material among products used for drainage structures. The recycled content of corrugated steel pipe, up to 96%, is rated as significant. Accordingly, specifying CSP can greatly assist in earning LEED® points in the "Material & Resources Credit 4" category. Additionally, steel is certified to meet specifications even with recycled material. While other drainage products may claim to be eco-friendly, care should be taken to ensure that these products conform to the AASHTO drainage pipe design and product specifications. Corrugated steel pipe, while having a high recycled content, conforms to and is routinely certified for compliance to AASHTO specifications. NCSPA members, on request, can also certify the recycle content for LEED's® qualification purposes.

Acceptance of Steel

Few if any construction materials are more universally recognized and accepted than steel. Steel has been used successfully in corrugated drainage structures for over 100 years throughout the world. For many years, corrugated steel pipe products have been included in the standard specifications of the American Association of State Highway and Transportation Officials (AASHTO), the American Society for Testing and Materials (ASTM), the Federal Highway Administration (FHWA), the American Railway Engineering and Maintenance-of-Way Association (AREMA), the Corps of Engineers (USACOE), the Federal Aviation Administration (FAA), the U.S. Forest Service (USFS), the Natural Resources Conservation Service (NRCS), the Canadian Standards Association (CSA), as well as state, county, township and municipal departments. The most recognized and respected consulting engineers worldwide specify corrugated steel pipe to meet design requirements.

DRAINAGE

Generally, drainage facilities can be classified as culverts, storm sewers, stream enclosures and bridges.

Culverts

The distinction between culverts and storm sewers is made primarily on the basis of length and type of inlets and outlets. A culvert is defined as a channel, serving as a continuation or a substitute for an open stream where that stream meets an artificial barrier such as a roadway, embankment, or levee. A culvert may be constructed in varying forms; round pipe, pipe arch, arch or box, and is usually less than 200 feet in length. CSP has dominated the culvert application market since it's emergence in the construction industry more than 100 years ago.

Multiple lines of CSP provide an effective low profile culvert crossing.

Storm Sewers

A storm sewer is a collection system for stormwater, surface water and street runoff, exclusive of domestic and industrial wastes. It is typically a series of tangent sections with manholes, inlets or bends at the junction points of the tangent sections. Stormwater is less, if at all, corrosive than rural watershed runoff. Erosion by stormwater flow through a CSP system is significantly less than in culverts.

As with CSP culverts, corrugated steel storm sewers have a service record of over 100 years. The strength, flexibility, positive joints and installation economies of steel storm sewers are assured by the use of rational corrosion design criteria and readily available coatings and linings. CSP storm sewers are also used to reline failing sewers of all sizes and shapes, with a minimum reduction in waterway area.

Many growing communities face the need for expansion in their storm sewer systems to accommodate residential, commercial and industrial development. Steel storm sewer pipe provides a ready solution. Its inherent long-term economic advantage to contractors and cost-conscious municipalities enables construction of projects that might not otherwise be built. Including CSP in the project specifications ensures these communities favorable bid prices, swift installation and a sound proven solution.

The use of corrugated steel pipe for storm sewers is common. Product data, design information and engineering considerations can be found in later chapters of this manual.

■ CSP storm sewers can be fabricated to fit any site.

Stream Enclosures

Large spans provided by CSP allow streams to be enclosed without disrupting the natural flow and stream bed. Often these enclosures protect the stream from new development or intrusions. Sizes, shapes and the capability with CSP to fabricate multiple angles allow the stream to take its natural course.

■ Structural plate structure following curvature of a natural stream.

Bridges and Bridge Replacements

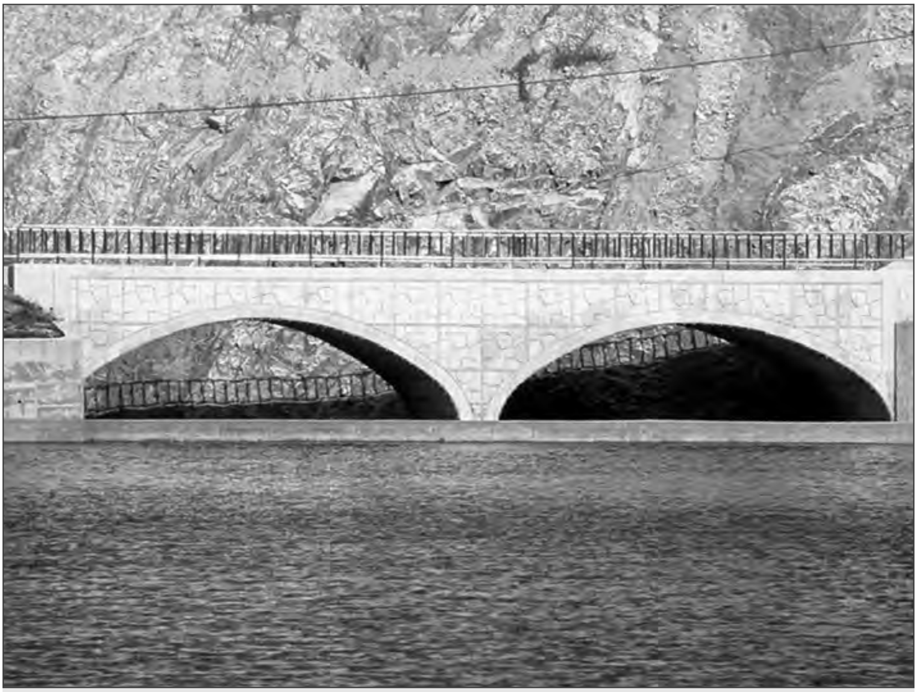
Currently, over 25% of the nation's bridges are structurally deficient. This situation is especially acute at county and municipal levels due to limited funds for maintenance and replacement on secondary roadways. Bridges can be economically replaced or rebuilt with corrugated steel structures; conventional corrugated steel pipe, structural plate pipe, pipe arches, arches and steel box culverts. The strength of corrugated steel allows spans that can exceed 80 feet, satisfying the majority of bridge installations.

■ CSP bridge replacement provides an economical installation with short road closure time.

CSP structures have been extremely popular for bridge replacement, providing the following advantages:

1. Economic value in production and installation costs.
2. Less design and construction time required than for conventional bridges, allowing earlier project completion.
3. Elimination of constant maintenance on bridge approaches and painting of superstructures.
4. No bridge deck deterioration problems.
5. Elimination of icy bridge deck problems.
6. Structures are readily available, shipped in one piece or easily field assembled.
7. In cases of future roadway expansion, lanes are easily widened by simply extending the ends of the structure.
8. Aesthetically and environmentally preferred, as they permit the use of a natural appearance in earth slope and vegetation.

For available shape configurations, sizes and waterway areas, see Chapter Two, Product Details and Fabrication. If an appropriate structure to meet project requirements is not shown, a design to meet specific site conditions can be provided by the manufacturers.



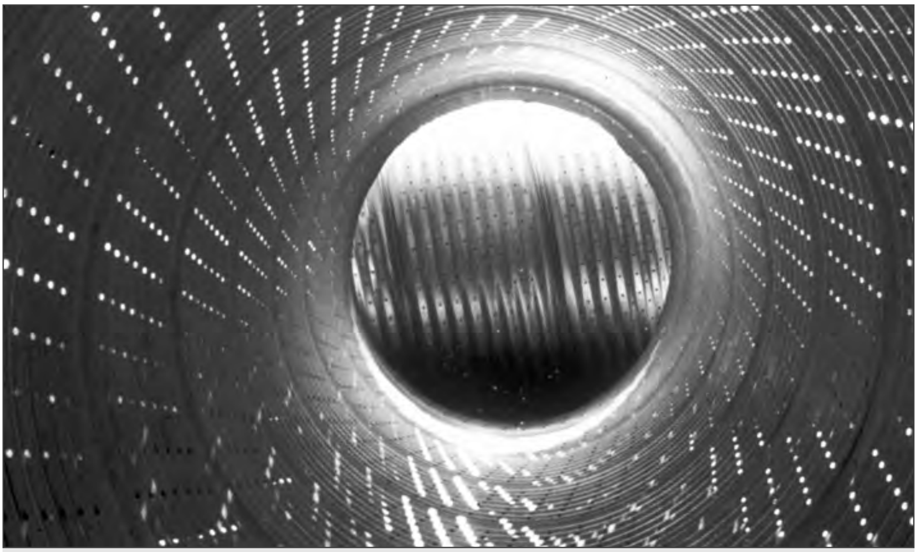
■ Low profile arch structural plate provides an aesthetic and economical stream crossing.

STORMWATER MANAGEMENT

The continuing spread of urbanization requires new drainage concepts to provide efficient and safe disposal of stormwater runoff. Existing storm drains in most areas cannot handle the additional volume at peak flow times. Severe flood damage can occur if good stormwater management tools, such as retention and detention systems, are not employed.

Retention Systems

Where stormwater runoff has no outlet for disposal, a retention system is a good solution. The stormwater is deliberately collected and stored, then allowed to dissipate by controlled infiltration into the ground. Additional benefits are found in the enhancement of ground water resources and filtration of stormwater through percolation. The use of fully perforated corrugated steel pipe for recharge wells and linear pipes is a very cost-effective way of disposing of unwanted stormwater.



Perforated CSP being used for underground retention system.

Detention Systems

In a situation where stormwater runoff has a restricted outlet due to downstream use, a detention system may be used. Temporary detention of stormwater in CSP storage tanks can be most economical and reliable. Stormwater is detained beyond the peak flow period and then is systematically released into the downstream storm drain. The demand for zero increase in the rate of runoff is very apparent in urban drainage design. Using corrugated steel pipe for retention and detention systems answers that need.

Site configuration adaptability and economical storage are accomplished with this detention system.

For further details on stormwater management covering retention/detention and surface disposal please refer to other chapters of this manual.

Sheet Flow Drainage

Intercepting sheet flow drainage at highway intersections, driveways and at the elevated shoulder of curves has become a critical element in highway design. For example, snow pushed to the high side shoulder on a curve melts as sunlight heats the pavement and shoulder. The runoff from the snow flows back across the roadway and freezes as evening temperatures fall. The result is sheet ice in a very critical area, creating a dangerous traffic hazard and repetitive maintenance problems.

An effective and economic solution to many sheet flow and pavement drainage problems can be a continuous longitudinal slotted drain corrugated steel pipe. A narrow slot in the top of the culvert intercepts runoff and the pipe carries it away as a part of the storm drain system. The system provides an inlet, runoff pipe, and grate all in one installation.

CSP Water Quality Structures

Corrugated steel pipe water quality structures can be used where it is necessary to control the quality of the runoff. These systems work in a three stage process; settling out solids, separating floating oils and solids, and filtering small fines. The CSP system can be built with a convenient means for access, allowing the structures to be cleaned and maintained periodically.

Effective water quality is attained by using an underground CSP sand filter.

Subdrainage

Subdrainage is the control of ground water in contrast to surface water or storm drainage. The civil engineer considers soil as an engineering construction material for building foundations, retaining wall backfills, embankments, cut sections for roads, highways and channels. Concern focuses on the basic soil characteristics, the presence of ground water, and whether subdrainage is practical for the soils on the project. Subdrainage can be a practical, economical method used in the stabilization of various construction sites: maintaining firm subgrades and structure foundations, eliminating wet cuts, preventing frost heave, preventing sloughing of fill and cut slopes, keeping recreational areas dry and reducing saturation of backfill behind retaining walls.

With a little study and experience, many soil and ground water problems can be recognized and solved with subdrainage pipe. For the more difficult cases, utilizing a soils engineer and laboratory are indispensable.

OTHER CSP DRAINAGE APPLICATIONS

Erosion Prevention

Soil erosion by water is a common and destructive force that plagues many project sites. It makes unsightly gullies on roadways, cut slopes and embankments. It gouges out side ditches, fills culverts with sediment and is a costly nuisance.

There are three basic ways of preventing erosion. The first is to treat the surface by paving, riprap, or use of erosion-resistant turf, vines, or other vegetation. Second, the velocity of the water may be reduced by means of ditch checks. The third method is to intercept the water through inlets and convey it in corrugated steel flumes, pipe spillways, stream enclosures, or storm drains. Larger streams may be controlled by steel sheeting, jetties, or retaining walls.

Corrugated steel pipe, with its long lengths, positive joints and flexibility to conform to shifting soil, provides a most dependable means of solving erosion problems.

Installation of a corrugated steel pipe spillway. Note the anti-seep collars on the pipe.

Dams and Levees

Earth dams, levees and many other types of embankments require culverts or outlets for intercepted or impounded water. Corrugated steel pipes are particularly advantageous and have enviable records for this type of service.

Dams and levees impound water and ultimately need pipe for drains and relief structures. Soil conditions at these locations are seldom ideal; therefore strong, flexible pipes are needed to resist disjointing, settlement and infiltration of the surrounding soil. CSP meets the demanding durability and structural needs for dams and levees. A local or regional office of the Natural Resource Conservation Service (NRCS) can be helpful in suggesting suitable details, such as diaphragms, anti-vortex baffles and riser sections based on proven local practice.

Fish Passage

In many sites, the need to accommodate migrating fish passage is an important consideration in culvert design. Transportation and drainage designers should seek early coordination with environmental, fish and wildlife agencies to insure that stream crossings that require provisions for fish passage are identified before design commences. Extensive experience has shown clearly that culverts can be designed to provide for fish passage. Design criteria for the specific fish species should be clarified during project development.

■ CSP arch preserving natural stream bed for fish passage.

Several variations in design are possible to accommodate fish passage:

Open-Bottom Culverts or arch-type culverts on spread footings permit the use of the natural stream bed. This approach is favored in streams with rock or erosion-resistant channels. Selection of a wider arch span provides for the maintenance of natural stream velocities during moderate flows.

Tailpond Control Weirs have proven to be the most practical approach to meet a minimum water depth requirement in the culvert barrel. A series of shallow weirs with a notch or small weir for low-flow passage have proved extremely effective. Larger weirs of more substantial design may require provision for separate fish ladder bypasses.

Oversized Culverts may be used when it is desirable to maintain a natural stream bed. Oversizing and depressing the culvert invert below the natural stream bed permits gravel and stone deposition, resulting in a nearly natural stream bed within the culvert. Numerous velocity profiles taken during floods indicate that wall and bed friction permit fish passage along the wall. In effect, the roughness of the steel barrel assists in fish passage.

Culverts with baffles attached to the invert may be used to control water velocities and depths. Considerable laboratory and field research has indicated that baffles or spoilers do significantly aid fish passage. The use of baffles in the barrel of a drainage structure may also be successful at sites where energy dissipation may be desirable. Additionally, baffles (sometimes called sill plates) will retain rock in a high velocity situation.

Multiple barrel installations have proven to be particularly effective in wide shallow streams. One barrel can be specifically designed with orifice plates inside the barrel to provide for fish passage.

Conversely, prevention of fish migration into upstream spawning grounds can also be accommodated through the incorporation of suitable weirs or barriers into the culvert design.

Power Plant Cooling Water Lines

Power plants require vast amounts of cooling water. Structural plate steel pipes over 18 feet in diameter have been used for water intake and outlets. These lines are typically subaqueous, requiring special underwater construction by divers. Corrugated steel is especially suitable for this type of construction and has been used for such lines in the Great Lakes region.

NONDRAINAGE APPLICATIONS

Steel conduits serve many practical purposes other than for drainage. Several of these are noted below.

Underpasses and Overpasses for Pedestrians and Animals

Pedestrian underpasses find their principal use in protecting people, including school children, who would otherwise be forced to cross hazards such as railway tracks, streets or highways.

Both domestic animals and wildlife benefit from the use of steel conduits as well. Frequently, large farms and ranches are divided by a highway or railroad, forcing livestock and native wildlife to make repeated dangerous crossings. An opening or underpass is often the most satisfactory solution to this problem. In some cases, large steel plate structures have been used to provide wildlife and ski trail crossings either over or under busy roads.

Safety is not the only advantage. Where a business, industry, or institution is divided by a busy street or railroad, a structural plate underpass is often the most convenient, direct and economical means of access.

■ Pedestrian and golf cart underpass.

Vehicular & Rail Grade Separations

Large underpasses serve as grade separations for automotive and railway traffic. For example, a county or local road can be carried under or over an interstate highway or railroad, often at less cost than by building a bridge. Smaller underpass structures are also used to provide passage for golf carts, snowmobiles, bicycles and other small vehicles.

■ Structural plate grade separation between railroad and highway.

Conveyors, Tunnels and Granaries

When a plant property is divided by a roadway or other barrier, a tunnel or an aerial bridging conduit may serve to join the property economically. In some cases a conveyor cover for short or long distance can serve to protect products from the elements while en route. Tracks, conveyor belts, or walkways may be used in these tunnels, bridging conduits, and conveyor covers.

■ Conveyor crossing and walkway access for aggregate handling facility.

Conveyor tunnels of heavy-wall thickness corrugated steel pipe are commonly used under storage piles of aggregates and other materials. Additionally, storage bins of heavy curved corrugated steel plates are used on construction jobs, as well as in plant material yards.

Utility Conduits

Water, steam and gas lines, sewers, or power cables must often pass between buildings or beneath embankments or other surface obstacles. Good engineering practice calls for placing them within a conduit to protect against direct loading, impact, corrosion, temperature extremes, and against sabotage or vandalism.

For encasing sewers or high pressure lines, a corrugated steel conduit helps minimize damage to the fill and surface installations caused by sudden breaks. A conduit large enough to walk through provides better access for inspections and repairs. Brackets, hangers, or cushioning bases are easily installed.

Utility conduits or tunnels may also double as air ducts. In the case of mines, munitions plants and other hazardous activities, these conduits may serve as ventilation overcasts and escapeways.

■ Corrugated steel utility conduit.

Bent Protection Systems

Corrugated steel pipe is utilized to reinforce and protect bridge piers, bents and foundations. Examples are applications such as roads and parking lots under elevated roadways or piles of aggregate from conveyors that accumulate against conveyor bents.

Bridge bent protection.

Containment Rings and Tanks

Corrugated steel pipe or structural plate rings in conjunction with a liner are used around storage vessels as a means to contain the contents of a spill. Similar structures are used to build large ponds for fish hatcheries. Many other applications in containment and storage can also be found.

Corrugated steel rings used for spill containment.

Agricultural Ventilation Systems

Corrugated steel pipe can be used in ventilation for corn, potato, onion, sugar beet and various grain stockpiles. In these cases, perforated CSP is placed under the stockpile. Air pumped through the pipe ventilates the stockpile, keeping the stored products fresh for longer periods of time.

CSP ventilation system for sugar beet storage facility.

Foundation Structures

Corrugated steel pipe has many uses in foundation applications; it can be used as encasement pipe in the construction of bridge piers, building foundations and towers. While these structures differ in end use and in construction methodology, CSP offers similar cost/benefit advantages for all of the above applications.

The primary advantage to CSP encasement is in the cost-control of concrete in the pier pour. Diameters commonly used range from 18 to 144 inches and lengths range from 7 to 75 feet. This type of installation technique is preferred in all but deep-water pier construction.

CSP form for wind turbine generator footing.

Aerial Conduits

Aerial conduits include at least two classes of structures. The first is exposed sewers, such as gravity water lines and service tunnels or bridges. The second class includes ducts for air and various gases used for ventilation or circulation. Aerial access bridges for safe movement of personnel or intra-plant materials handling have also proved useful.

Service Tunnels

Rather than ground level crossings or subterranean passageways, aerial conduits can be a good choice in industry. Bridging between adjacent buildings of a manufacturing plant may be desirable for more direct access for employees, materials, finished products, or utility lines. Other applications are seen at mine tipples, quarries, or docks, where the aerial lines may be quite lengthy.

Architectural Design

Corrugated steel pipe has clean symmetrical lines that have long drawn architects to apply CSP both in aesthetic and structural uses. Common uses are supports and free standing columns.

CSP entry columns for commercial building.

Ventilation and Fan Ducts

Mining, industry and construction operations require various degrees of ventilation to protect against health hazards arising from toxic gases, excessive heat, moisture, dust and possible explosions. Ventilation conduits or fan ducts extend from the ventilating fan to the portal of the fresh air tunnel or air shaft. Corrugated steel pipe ducts have been widely used for years due to their high strength-to-weight ratio. Further, they are fully salvable if a change of operations is necessary. They resist destruction from explosions, are fire resistant, and contribute to mine safety through confining explosion and fire in the event of disaster.

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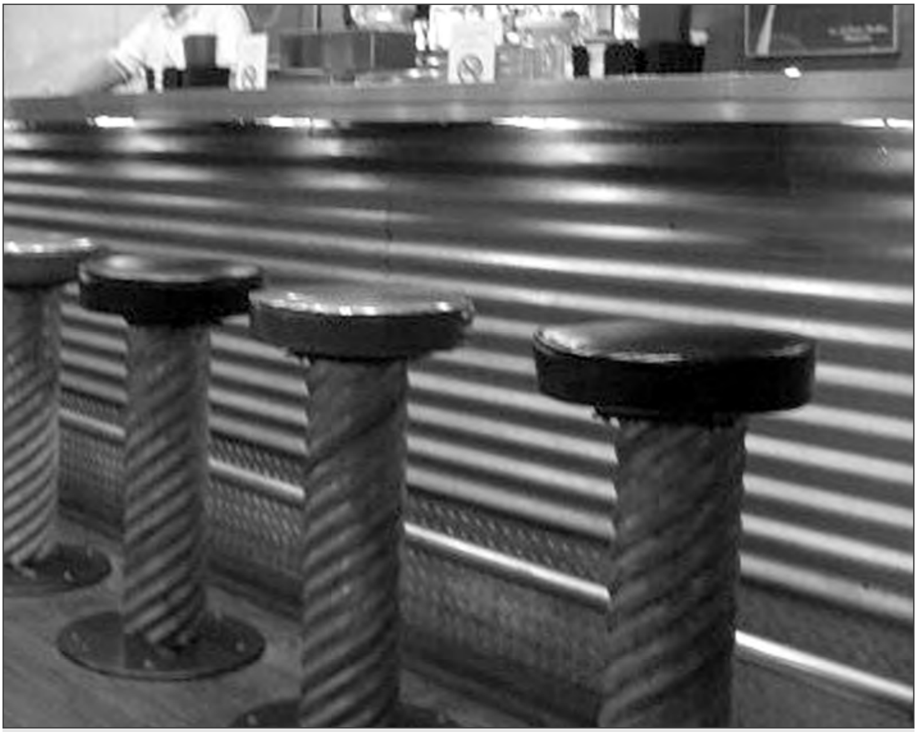
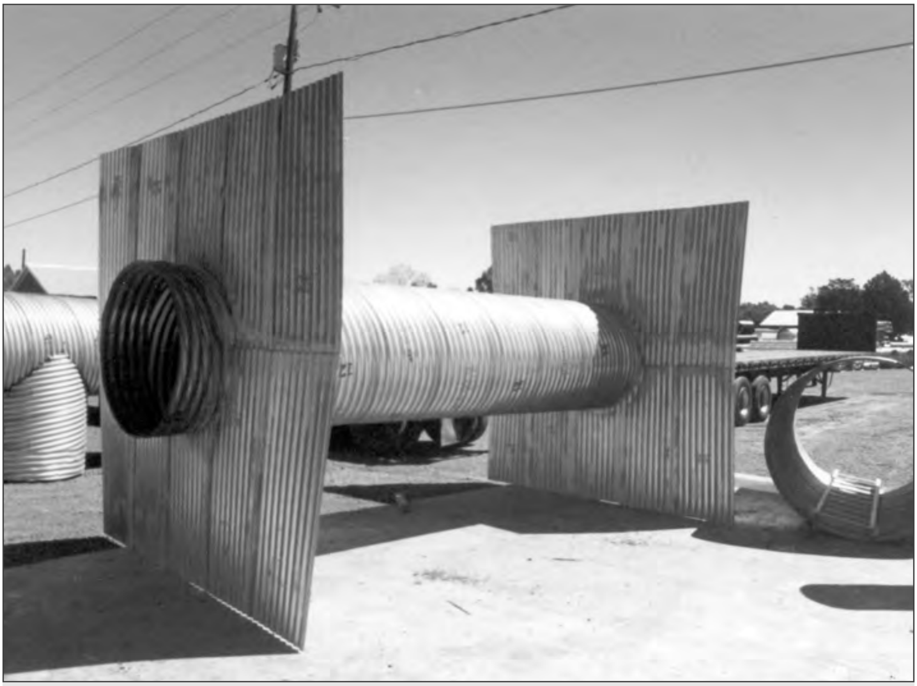
AREMA, *Engineering Manual*, American Railway Engineering and Maintenance-of-Way Association, 8201 Corporate Drive, Ste. 1125, Landover, MD 20785-2230.

ASTM, *Annual Book of ASTM Standards*, Coated Steel Products, Vol. 01.06,, American Society for testing and Materials, 100 Barr Harbor Drive, PO Box C700, West Conshohocken, PA 19428-2959.



CSP and structural plate used in interior design.

A section of CSP used for an entry way.



Examples of the flexibility of CSP fabrication.

CHAPTER two

Product Details and Fabrication

INTRODUCTION

The application of corrugated steel pipe to the solution of various drainage problems has been described and illustrated in Chapter 1. These products are applicable to a wide segment of the construction industry, including highways, railways, streets, urban areas, airports, industrial and commercial development, flood control and conservation.

The steel products presented in this CSP Design Manual may be available at locations around the world, but not all products are fabricated in the USA. The NCSPA prepared this manual with full knowledge that highway projects funded by the FHWA require inclusion of the "Buy America" clause in the contract documents and the use of US-Made steel in the project. For FHWA funded projects, designers and specifiers should verify the origin and availability of CSP products through contact with local corrugated steel pipe and plate fabricators.

NCSPA does not accept responsibility for the designer's selection of material for drainage applications, but encourages the designer to evaluate the numerous corrugated steel products that meet the requirements of their project. It is suggested that the designer check the NCSPA website at www.NCSPA.org for additional technical guidance related to the selection of drainage pipe and structures.

The examples presented in this design manual are not all-inclusive or complete solutions, they are intended only to show the adaptability and wide acceptance of one material—corrugated steel—in providing the solution to some of the problems facing the design engineer.

So vast are the annual expenditures for construction, that the skills of resourceful qualified engineers are required to research (analyze), select, design and apply the available materials and products that most economically serve their purpose. Mass transportation, anti-pollution facilities, flood protection and other related construction, conceivably can require drainage facilities in comparable measure. The need for carefully considering the economics of providing and maintaining these facilities is obvious.

DESIGN FACTORS

Drainage design begins with reconnaissance and location surveys. The services of experienced soils and drainage engineers are the best assurance of economical construction and subsequent minimum maintenance.

The following design factors must be considered:

1. Size, shape, alignment, grade and other pipe details depend on hydrology, hydraulics, site conditions and service requirements. (See Chapters 3, 4 and 5.)

2. Structural adequacy to meet embankment and superimposed live loads, along with hydraulic forces. (See Chapters 7 and 8.)
3. Trouble-free service through selection of materials to resist abrasion and assure long term durability. (See Chapter 9.)
4. Economics—first cost of materials, installation cost, maintenance cost over the life of the pipe. (See Chapter 11.)

In addition to these, the design engineer can make a value-analysis of such other factors as: suitable sources of supply, probable delivery schedule, influence of climate or season of year, coordination with other construction schedules, supplier's assistance, and ease of repair or replacement in relation to the importance or service of the installation.

Alternative materials and designs should be considered so that the final selection will provide the most economical and satisfactory solution for the overall installation and its users.

BACKGROUND

Corrugating a flat sheet has long been known to increase its stiffness and strength Corrugated steel sheets have been produced almost since the first rolling mill was built in England in 1784. But it was not until after 1890, when mass-produced steel sheets became abundant, that their use grew rapidly.

Corrugated steel pipe was first developed and used for culverts in 1896. As experience was gained in the use of this thin-wall, lightweight, shop-fabricated pipe, the diameters gradually increased to 96 inches and larger. Fill heights became greater, even exceeding 100 feet A further development, in 1931, was structural plate pipe with larger corrugations, for field assembly. Diameters and arch spans beyond 26 feet have been installed successfully.

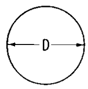
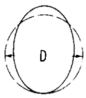
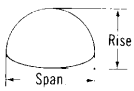
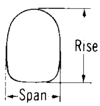
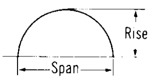
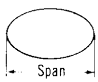
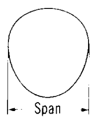
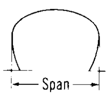
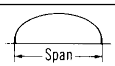
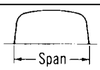
SHAPES

The designer has a wide choice of standard cross-sectional shapes of corrugated steel and structural plate pipe as shown in Table 2.1. Size planned and site conditions use may control the shape selected, with strength and economy as additional factors. For sectional properties of corrugated steel sheets and plates, see Tables 2.3 through 2.15. For seam strengths, sizes, weights and other details, see Tables 2.16 through 2.49.

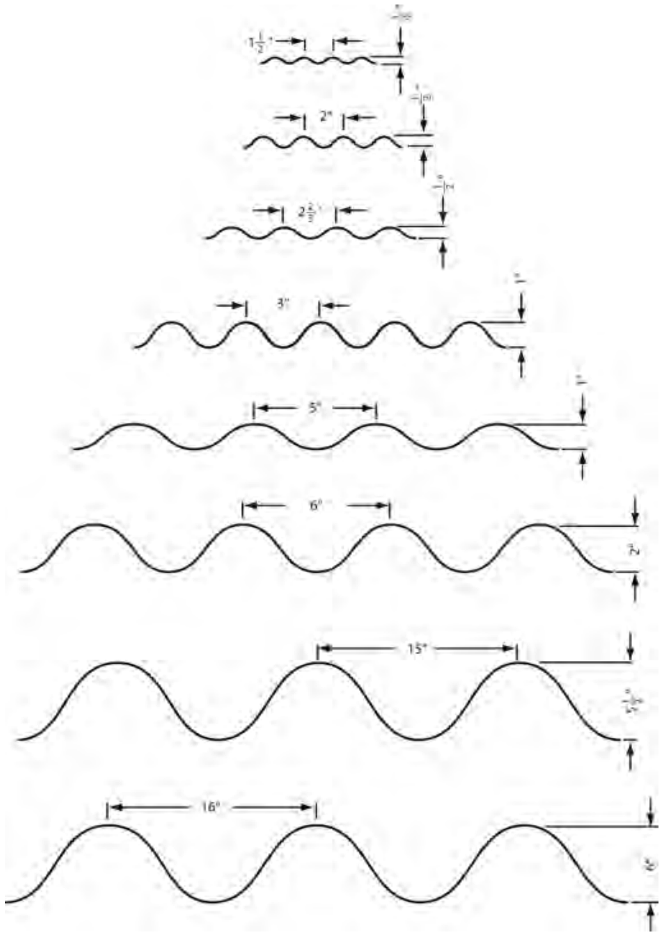
There are many kinds of corrugations, some of which are shown in Fig. 2.1 and 2.2. Corrugations profiles commonly used for pipes or conduits consist of circular arcs and alternating tangent segments or alternating rectangular ribs and flat segments. Corrugation profiles are typically described by pitch, depth and inside forming radius. Pitch is measured at right angles to the corrugations from crest to crest. The corrugation profiles shown in Figures 2.1 and 2.2 are not fabricated by every CSP manufacturer. Check with your local fabricator before specifying a corrugation profile.

Table 2.1

Shapes and uses of corrugated conduits

Shape		Range of Sizes	Common Uses
Round		6 in. - 51 ft	Culverts, subdrains, sewers,service tunnels, etc. All plates same radius. For medium and high fills (or trenches).
Vertical ellipse 5% nominal		4 - 21 ft nominal; before elongating	Culverts, sewers, service tunnels, recovery tunnels. Plates of varying radii; shop fabrication. For appearance and where backfill compaction is only moderate.
Pipe Arch		Span x Rise 17 in. x 13 in. to 20 ft 7 in x 13 ft 2 in.	Where headroom is limited. Has hydraulic advantages at low flows. Corner plate radius. 18 inches or 31 inches for structural plate.
Underpass*		Span x Rise 5 ft 8 in. x 5 ft 9 in. to 20 ft 4 in. x 17 ft 9 in.	For pedestrians, livestock or vehicles (structural plate).
Arch		Span x Rise 5 ft x 1 ft 9 1/2 in. to 82 ft x 42 ft	For low clearance large waterway opening, and aesthetics (structural plate).
Horizontal Ellipse		Span 7 - 40 ft	Culverts, grade separations, storm sewers, tunnels (structural plate).
Pear		Span 25 - 30 ft	Grade separations, culverts, storm sewers, tunnels (structural plate).
High Profile Arch		Span 20 - 83 ft	Culverts, grade separations, storm sewers and tunnels. Ammunition magazines, earth covered storage (structural plate).
Low Profile Arch		Span 20 - 83 ft	Low-wide waterway enclosures, culverts, storm sewers (structural plate).
Box Culverts		Span 10 - 53 ft	Low-wide waterway enclosures, culverts, storm sewers (structural plate).
Specials		Various	For lining old structures or other special purposes. Special fabrication.

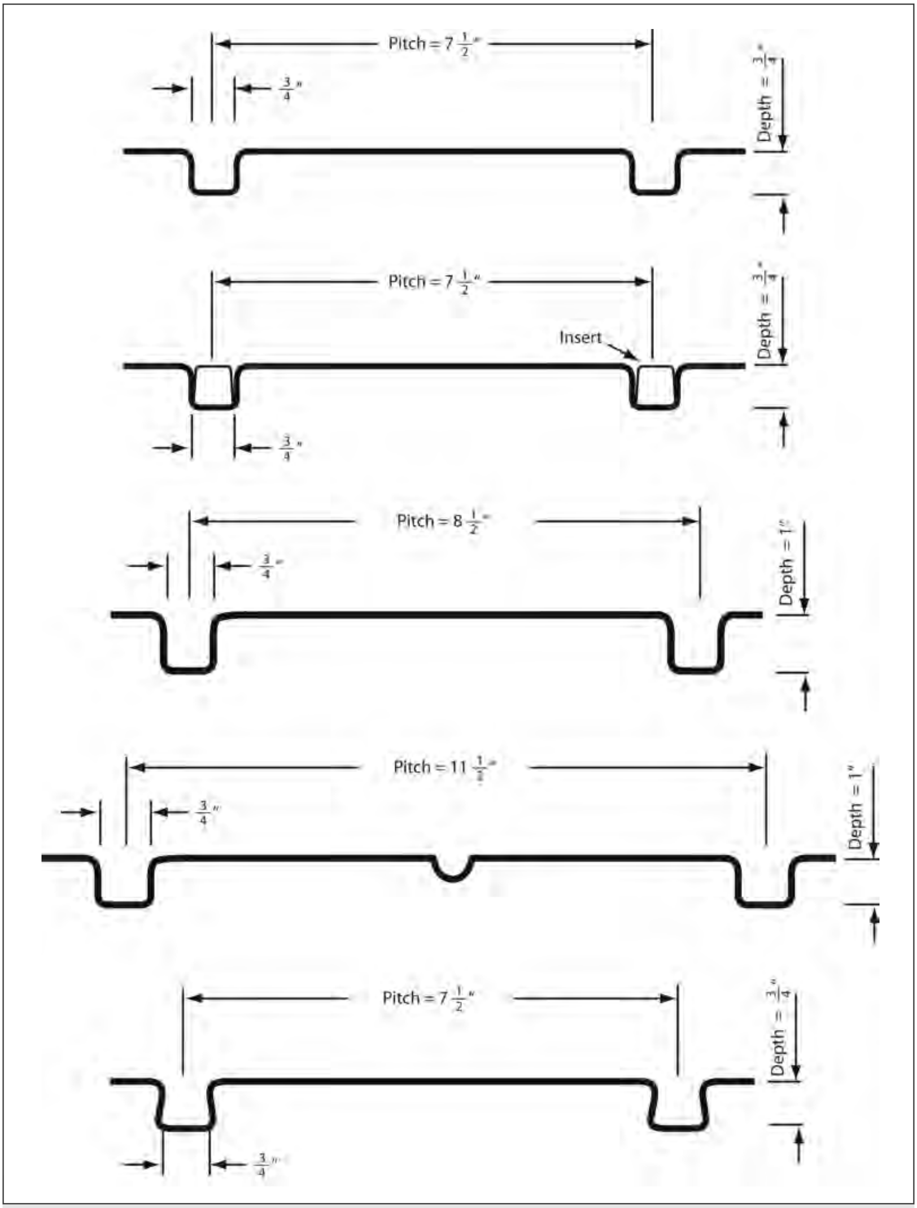
Notes: *For equal area or clearance, the round shape is generally more economical and easier to assemble.



■ **Figure 2.1** Arc and tangent corrugations.

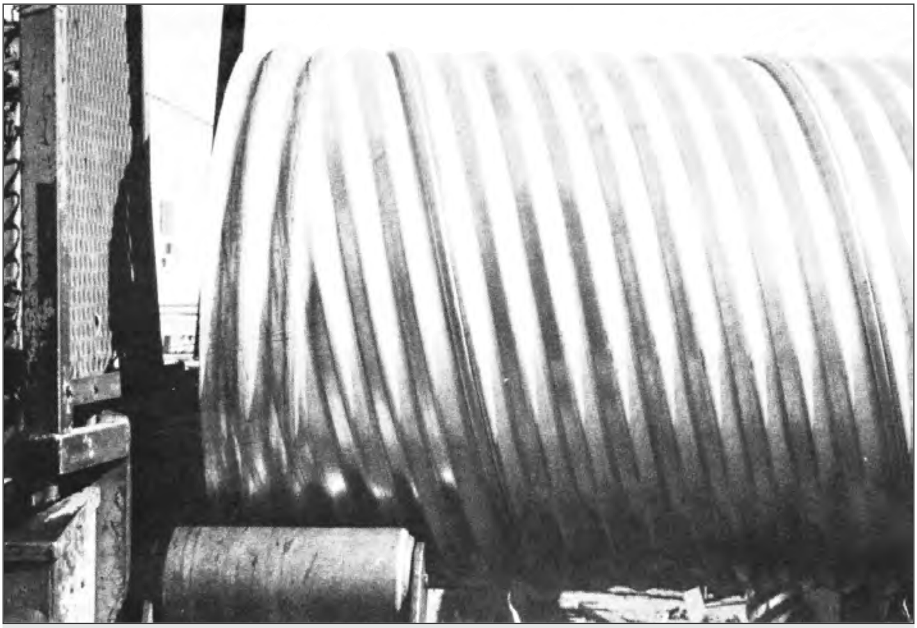
For riveted or resistance spot-welded pipe with circumferential (annular) seams, the corrugations are of 2 2/3 inches pitch by 1/2 inch depth or 3 inches by 1 inch. For lock seam pipe, the seams and corrugations run helically (or spirally) around the pipe. For small diameter subdrain pipe (6, 8, 10 inches, etc.) the pitch vs. depth dimension is 1 1/2 x 1/4 inches. Larger sizes (diameters to 144 inches depending on profile) use 2 x 1/2 inch, 2 2/3 x 1/2 inch, 3 x 1 inch, and 5 x 1 inch corrugations.

The most recent lock seam corrugations introduced to the market were the spiral rib profiles. Developed in the mid 1980's, the pipe wall is spirally formed using rectangular formed ribs between flat wall areas. This unique profile configuration was developed to provide flow characteristics equal to those piping systems normally considered smooth wall. Three profile configurations are available – 3/4 inch x 3/4 inch x 7 1/2 inches, 3/4 inch



■ **Figure 2.2** Spiral rib corrugations.

x 1 inch x 8 1/2 inches and 3/4 inch x 1 inch x 11 1/2 inches (covering diameters from 18 through 108 inches). Structural plate pipe consists of corrugated sheets that are bolted together to form the required shape. The 6 x 2 inch corrugation was the original structural plate corrugation profile. The most recent corrugation profiles introduced for structural plate are commonly referred to as 'deep' corrugated. Corrugation profiles for 'deep' corrugated structural plate include 15 x 5 1/2 inch and the 16 x 6 inch corrugations.



Rerolling an annular end on helical corrugated pipe.

Corrugated steel pipe nested for shipment.

SECTIONAL PROPERTIES

Sectional properties of the arc-and-tangent type of corrugation are derived mathematically. These include area, A, moment of inertia, I, section modulus, S, and radius of gyration, r. Research by the American Iron and Steel Institute has shown that failure loads in bending and deflection within the elastic range can be closely predicted by using computed sectional properties of the corrugated sheet. See Tables 2.3 through 2.15.

Table 2.2

Conversion of nominal gage to thickness

Gage No.	22	20	18	16	14	12	10
Uncoated Thickness (in.)	0.0299	0.0359	0.0478	0.0598	0.0747	0.1046	0.1345
Galvanized Thickness* (in.)	0.034	0.040	0.052	0.064	0.079	0.109	0.138
Galvanized Structural Plate Thickness (in.)						0.111	0.140
Gage No.	**8**	**7**	**5**	**3**	**1**	**5/16"**	**3/8"**
Uncoated Thickness (in.)	0.1644	0.1838	0.2145	0.2451	0.2758	0.3125	0.3750
Galvanized Thickness* (in.)	0.168						
Galvanized Structural Plate Thickness (in.)	0.170	0.188	0.218	0.249	0.280	0.318	0.380

Notes: * Also referred to as specified thickness for corrugated steel pipe products.
For structural plate, tunnel liner plates and other products, see chapters on those products.

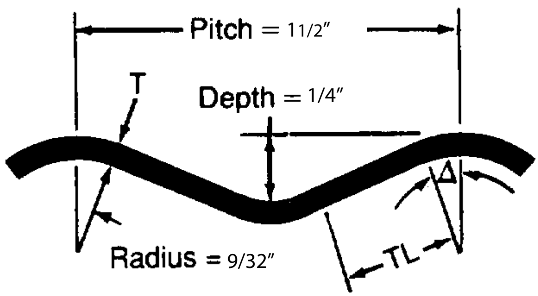


Table 2.3

Sectional properties of 11/2 x 1/4 in. (Helical)

Specified Thickness	Uncoated Thickness *T*	Area of Section *A*	Tangent Length *TL*	Tangent Angle *Δ*	Moment of Inertia *I*	Section Modulus *S*	Radius of Gyration *r*	Developed Width Factor
(in.)	(in.)	(in.^{2}/ft)	(in.)	(Degrees)	(in.^{4}/in)	(in.^{3}/ft)	(in.)	
0.040*	0.0359	0.456	0.571	21.44	0.00025	0.0213	0.0816	1.060
0.052	0.0478	0.608	0.566	21.52	0.00034	0.0277	0.0842	1.060
0.064	0.0598	0.761	0.560	21.61	0.00044	0.0340	0.0832	1.060
0.079	0.0747	0.950	0.554	21.71	0.00057	0.0419	0.0846	1.060
0.109*	0.1046	1.331	0.540	21.94	0.00086	0.0580	0.0879	1.060
0.138*	0.1345	1.712	0.526	22.17	0.00121	0.0753	0.0919	1.061
0.168*	0.1644	2.093	0.511	22.42	0.00163	0.0945	0.0967	1.061

* Thickness not commonly available. Information only.
Notes: 1. Per foot of projection about the neutral axis.
To obtain *A* or *S* per *inch* of width, divide the above values by 12.
2. Developed width factor measures the increase in profile length due to corrugating.
Dimensions are subject to manufacturing tolerances.

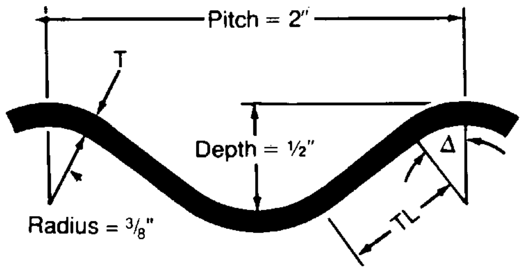
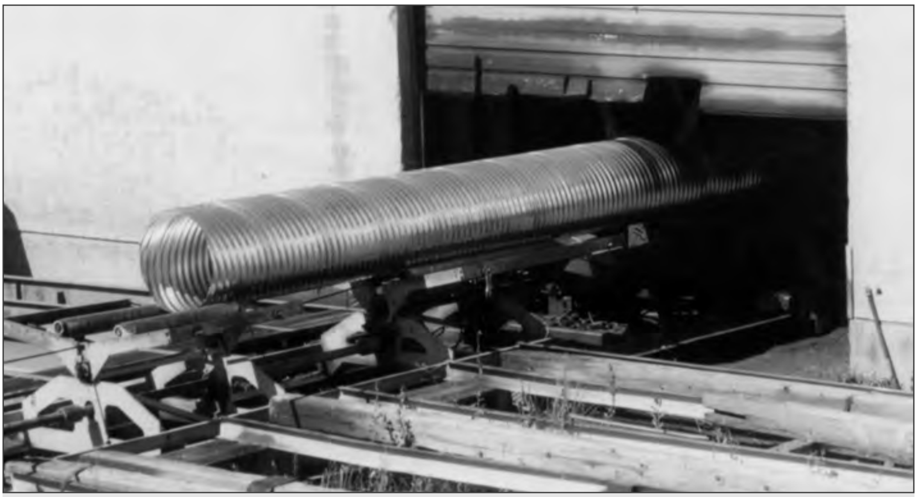


Table 2.4

Sectional properties of 2 x 1/2 in. (Helical)

Specified Thickness	Uncoated Thickness T	Area of Section A	Tangent Length TL	Tangent Angle Δ	Moment of Inertia I	Section Modulus S	Radius of Gyration r	Developed Width Factor
(in.)	(in.)	(in.^{2}/ft)	(in.)	(Degrees)	(in.^{4}/in)	(in.^{3}/ft)	(in.)	
0.040*	0.0359	0.489	0.681	33.12	0.0011	0.0513	0.1676	1.136
0.052	0.0478	0.652	0.672	33.29	0.0015	0.0673	0.1682	1.136
0.064	0.0598	0.815	0.663	33.46	0.0019	0.0832	0.1690	1.136
0.079	0.0747	1.019	0.625	33.68	0.0025	0.1025	0.1700	1.137
0.109	0.1046	1.428	0.629	34.13	0.0035	0.1406	0.1725	1.138
0.138*	0.1345	1.838	0.605	34.62	0.0047	0.1783	0.1754	1.139
0.168*	0.1644	2.249	0.579	35.13	0.0060	0.2166	0.1788	1.140

* Thickness not commonly available. Information only.
Notes: 1. Per foot of projection about the neutral axis.
To obtain ***A*** or ***S*** per ***inch*** of width, divide the above values by 12.
2. Developed width factor measures the increase in profile length due to corrugating.
Dimensions are subject to manufacturing tolerances.



Manufacturing of corrugated steel pipe.

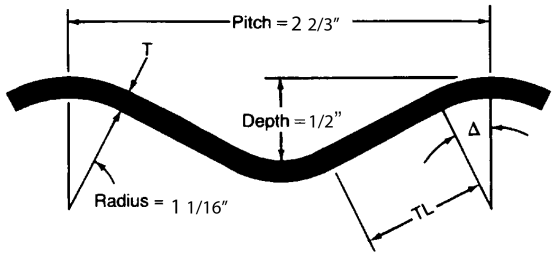


Table 2.5

Sectional properties of 2 2/3 x 1/2 in. (Annular or Helical)

Specified Thickness	Uncoated Thickness *T*	Area of Section *A*	Tangent Length *TL*	Tangent Angle *Δ*	Moment of Inertia *I*	Section Modulus *S*	Radius of Gyration *r*	Developed Width Factor
(in.)	(in.)	($in.^2$/ft)	(in.)	(Degrees)	($in.^4$/in)	($in.^3$/ft)	(in.)	
0.040*	0.0359	0.465	0.785	26.56	0.0011	0.0503	0.1702	1.080
0.052	0.0478	0.619	0.778	26.65	0.0015	0.0659	0.1707	1.080
0.064	0.0598	0.775	0.770	26.74	0.0019	0.0812	0.1712	1.080
0.079	0.0747	0.968	0.760	26.86	0.0024	0.0998	0.1721	1.080
0.109	0.1046	1.356	0.740	27.11	0.0034	0.1360	0.1741	1.080
0.138	0.1345	1.744	0.720	27.37	0.0045	0.1714	0.1766	1.081
0.168	0.1644	2.133	0.699	27.65	0.0057	0.2069	0.1795	1.081

* Thickness not commonly available. Information only.
Notes: 1. Per foot of projection about the neutral axis.
To obtain *A* or *S* per *inch* of width, divide the above values by 12.
2. Developed width factor measures the increase in profile length due to corrugating.
Dimensions are subject to manufacturing tolerances.

Modern helical lock seam shop fabrication is the most common method of manufacturing corrugated steel pipe.

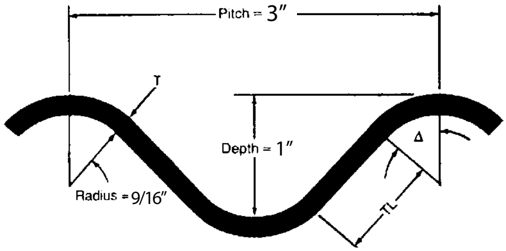


Table 2.6

Sectional properties of 3 x 1 in. (Annular or Helical)

Specified Thickness	Uncoated Thickness *T*	Area of Section *A*	Tangent Length *TL*	Tangent Angle *Δ*	Moment of Inertia *I*	Section Modulus *S*	Radius of Gyration *r*	Developed Width Factor
(in.)	(in.)	(in.²/ft)	(in.)	(Degrees)	(in.⁴/in)	(in.³/ft)	(in.)	
0.040*	0.0359	0.534	0.963	44.19	0.0052	0.1194	0.3403	1.239
0.052	0.0478	0.711	0.951	44.39	0.0069	0.1578	0.3410	1.240
0.064	0.0598	0.890	0.938	44.60	0.0087	0.1961	0.3417	1.240
0.079	0.0747	1.113	0.922	44.87	0.0109	0.2431	0.3427	1.241
0.109	0.1046	1.560	0.889	45.42	0.0146	0.3358	0.3448	1.243
0.138	0.1345	2.008	0.855	46.02	0.0202	0.4269	0.3472	1.244
0.168	0.1644	2.458	0.819	46.65	0.0251	0.5170	0.3499	1.246

* Thickness not commonly available. Information only.
Notes: 1. Per foot of projection about the neutral axis.
To obtain ***A*** or ***S*** per ***inch*** of width, divide the above values by 12.
2. Developed width factor measures the increase in profile length due to corrugating.
Dimensions are subject to manufacturing tolerances.

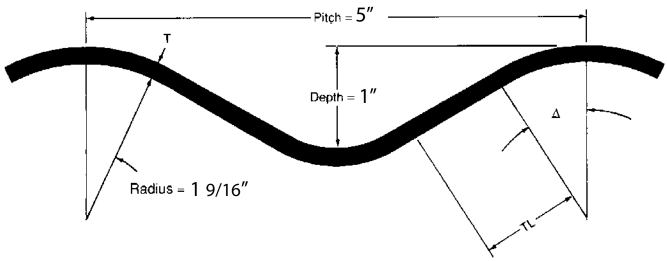


Table 2.7

Sectional properties of 5 x 1 in. (Helical)

Specified Thickness	Uncoated Thickness *T*	Area of Section *A*	Tangent Length *TL*	Tangent Angle *Δ*	Moment of Inertia *I*	Section Modulus *S*	Radius of Gyration *r*	Developed Width Factor
(in.)	(in.)	(in.²/ft)	(in.)	(Degrees)	(in.⁴/in)	(in.³/ft)	(in.)	
0.064	0.0598	0.794	0.730	35.58	0.0089	0.1960	0.3657	1.106
0.079	0.0747	0.992	0.708	35.80	0.0111	0.2423	0.3663	1.107
0.109	0.1046	1.390	0.664	36.30	0.0156	0.3330	0.3677	1.107
0.138	0.1345	1.788	0.616	36.81	0.0203	0.4210	0.3693	1.108
0.168	0.1644	2.186	0.564	37.39	0.0250	0.5069	0.3711	1.108

Notes: 1. Per foot of projection about the neutral axis. To obtain ***A*** or ***S*** per ***inch*** of width, divide the above values by 12.
2. Developed width factor measures the increase in profile. Dimensions are subject to manufacturing tolerances.
3. Actual Pitch = 4.9213 in. and Actual Depth = 1.0236 in. Dimensions shown on sketch are nominal.

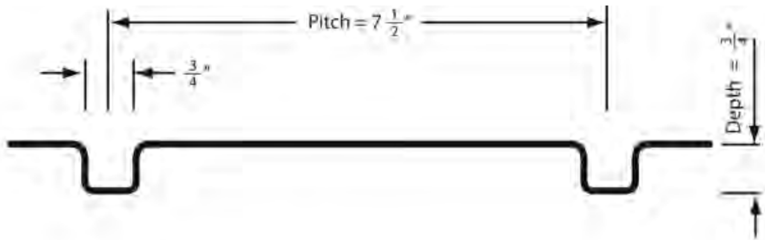


Table 2.8

Effective sectional properties of 3/4 x 3/4 x 7 1/2 in. spiral rib (Helical)

Specified Thickness	Uncoated Thickness *T*	Area of Section *A*	Moment of Inertia *I*	Section Modulus *S*	Radius of Gyration *r*	Developed Width Factor
(in.)	(in.)	(in.^{2}/ft)	(in.^{4}/in.)	(in.^{3}/ft)	(in.)	
0.064	0.0598	0.509	0.0028	0.0747	0.258	1.170
0.079	0.0747	0.712	0.0037	0.0940	0.250	1.168
0.109	0.1046	1.184	0.0055	1.1326	0.237	1.165
0.138	0.1345	1.717	0.0074	0.1706	0.228	1.162

Notes:
1. Per foot of projection about the neutral axis. To obtain ***A*** or ***S*** per ***inch*** of width, divide the above values by 12.
2. Developed width factor measures the increase in profile length due to corrugating. Dimensions are subject to manufacturing tolerances.
3. Properties are effective section properties at full yield stress.

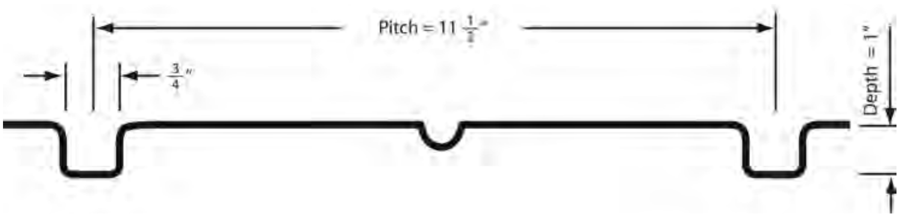


Table 2.9

Effective sectional properties of 3/4 x 1 x 11 1/2 in. spiral rib (Helical)

Specified Thickness	Uncoated Thickness *T*	Area of Section *A*	Moment of Inertia *I*	Section Modulus *S*	Radius of Gyration *r*	Developed Width Factor
(in.)	(in.)	(in.^{2}/ft)	(in.^{4}/in.)	(in.^{3}/ft)	(in.)	
0.064	0.0598	0.374	0.0046	0.0736	0.383	1.154
0.079	0.0747	0.524	0.0061	0.0931	0.373	1.153
0.109	0.1046	0.883	0.0093	0.1324	0.355	1.151

Notes:
1. Per foot of projection about the neutral axis. To obtain ***A*** or ***S*** per ***inch*** of width, divide the above values by 12.
2. Developed width factor measures the increase in profile length due to corrugating. Dimensions are subject to manufacturing tolerances.
3. Properties are effective section properties at full yield stress.

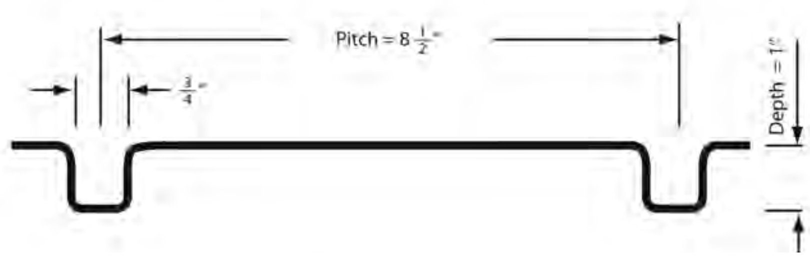


Table 2.10

Effective sectional properties of 3/4 x 1 x 8 1/2 in. spiral rib (Helical)

Specified Thickness	Area of Section *A*	Moment of Inertia *I*	Section Modulus *S*	Radius of Gyration *r*	Developed Width Factor
(in.)	(in.^{2}/ft)	(in.^{4}/in.)	(in.^{3}/ft)	(in.)	
0.064	0.499	0.0060	0.0957	0.379	1.199
0.079	0.694	0.0079	0.1210	0.370	1.198
0.109	1.149	0.0120	01719	0.354	1.944

Notes:
1. Per foot of projection about the neutral axis. To obtain *A* or *S* per *inch* of width, divide the above values by 12.
2. Developed width factor measures the increase in profile length due to corrugating. Dimensions are subject to manufacturing tolerances.
3. Properties are effective section properties at full yield stress.

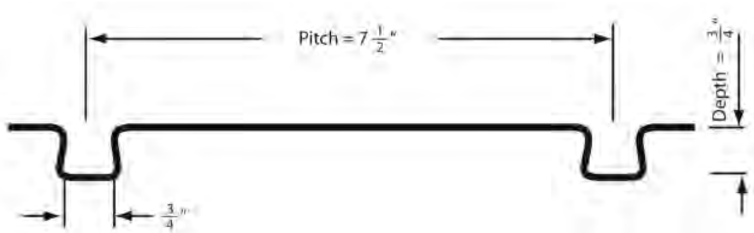


Table 2.11

Effective sectional properties of 3/4 x 3/4 x 7 1/2 in. composite ribbed steel pipe (Helical)

Specified Thickness	Uncoated Thickness *T*	Area of Section *A*	Moment of Inertia *I*	Section Modulus *S*	Radius of Gyration *r*	Developed Width Factor
(in.)	(in.)	(in.^{2}/ft)	(in.^{4}/in.)	(in.^{3}/ft)	(in.)	
0.064	0.0598	0.520	0.0028	0.0643	0.253	1.239
0.079	0.0747	0.728	0.0036	0.0817	0.245	1.233
0.109	0.1046	1.212	0.0054	0.1174	0.232	1.216
0.138	0.1345	1.758	0.0073	0.1541	0.223	1.199

Notes:
1. Per foot of projection about the neutral axis. To obtain *A* or *S* per *inch* of width, divide the above values by 12.
2. Developed width factor measures the increase in profile length due to corrugating. Dimensions are subject to manufacturing tolerances.
3. Properties are effective section properties at full yield stress.

Perforated corrugated steel pipe is widely used for through-the-pile ventilation of perishable crops.

Multiple CSP lines form an underground stormwater storage facility.

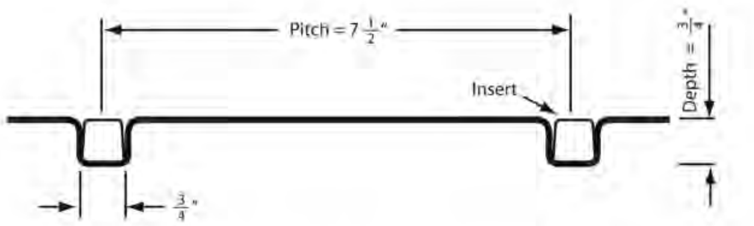


Table 2.12

Effective sectional properties of 3/4 x 3/4 x 7 1/2 in. spiral rib with insert (Helical)

Specified Thickness	Uncoated Thickness *T*	Area of Section *A*	Moment of Inertia *I*	Section Modulus *S*	Radius of Gyration *r*	Developed Width Factor
(in.)	(in.)	(in.^{2}/ft)	(in.^{4}/in.)	(in.^{3}/ft)	(in.)	
0.064	0.0598	0.509	0.0028	0.0747	0.258	1.170
0.079	0.0747	0.712	0.0037	0.0940	0.250	1.168
0.109	0.1046	1.184	0.0055	1.1326	0.237	1.165
0.138	0.1345	1.717	0.0074	0.1706	0.228	1.162

Notes: 1. Per foot of projection about the neutral axis. To obtain ***A*** or ***S*** per ***inch*** of width, divide the above values by 12.
2. Developed width factor measures the increase in profile length due to corrugating. Dimensions are subject to manufacturing tolerances.
3. Properties are effective section properties at full yield stress.

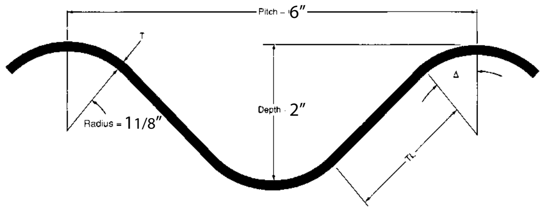


Table 2.13

Sectional properties of 6 x 2 in. (Annular)

Specified Thickness	Uncoated Thickness *T*	Area of Section *A*	Tangent Length *TL*	Tangent Angle *Δ*	Moment of Inertia *I*	Section Modulus *S*	Radius of Gyration *r*	Developed Width Factor
(in.)	(in.)	(in.^{2}/ft)	(in.)	(Degrees)	(in.^{4}/in)	(in.^{3}/ft)	(in.)	
0.111	0.1046	1.556	1.893	44.47	0.0604	0.689	0.682	1.240
0.140	0.1345	2.003	1.861	44.73	0.0782	0.879	0.684	1.241
0.170	0.1644	2.449	1.828	45.00	0.0962	1.066	0.686	1.242
0.188	0.1838	2.739	1.807	45.18	0.1080	1.187	0.688	1.242
0.218	0.2145	3.199	1.773	45.47	0.1269	1.376	0.690	1.243
0.249	0.2451	3.658	1.738	45.77	0.1462	1.562	0.692	1.244
0.280	0.2758	4.119	1.702	46.09	0.1658	1.749	0.695	1.245
0.318	0.3125	4.671	1.653	46.47	0.1900	1.968	0.698	1.246
0.380	0.3750	5.613	1.581	47.17	0.2320	2.340	0.704	1.247

Notes: 1. Per foot of projection about the neutral axis.
To obtain ***A*** or ***S*** per ***inch*** of width, divide the above values by 12.
2. Developed width factor measures the increase in profile length due to corrugating.
Dimensions are subject to manufacturing tolerances.

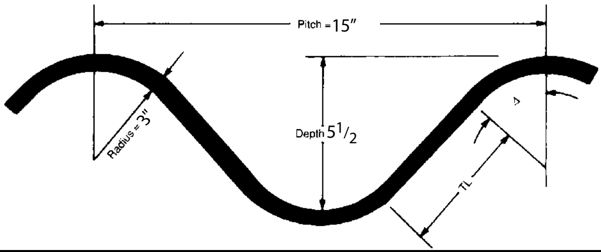


Table 2.14

Sectional properties of 15 x 5 1/2 in. (Annular)

Specified Thickness	Uncoated Thickness *T*	Area of Section *A*	Tangent Length *TL*	Tangent Angle *Δ*	Moment of Inertia *I*	Section Modulus *S*	Radius of Gyration *r*	Developed Width Factor
(in.)	(in.)	($in.^2$/ft)	(in.)	(Degrees)	($in.^4$/in)	($in.^3$/ft)	(in.)	
0.140	0.1345	2.260	4.361	49.75	0.7146	2.8406	1.9481	1.400
0.170	0.1644	2.762	4.323	49.89	0.8746	3.4602	1.9494	1.400
0.188	0.1838	3.088	4.299	49.99	0.9786	3.8599	1.9502	1.400
0.218	0.2145	3.604	4.259	50.13	1.1436	4.4888	1.9515	1.400
0.249	0.2451	4.118	4.220	50.28	1.3084	5.1114	1.9527	1.400
0.280	0.2758	4.633	4.179	50.43	1.4722	5.7317	1.9540	1.400
0.193	0.1875	3.150	4.293	50.00	0.9985	3.9359	1.9503	1.400
0.255	0.2500	4.200	4.213	50.31	1.3349	5.2107	1.9529	1.400
0.318	0.3125	5.250	4.131	50.62	1.6730	6.4678	1.9555	1.400
0.380	0.3750	6.300	4.047	50.94	2.0128	7.7076	1.9580	1.400

Notes: 1. Per foot of projection about the neutral axis.
To obtain *A* or *S* per *inch* of width, divide the above values by 12.
2. Developed width factor measures the increase in profile length due to corrugating.
Dimensions are subject to manufacturing tolerances.

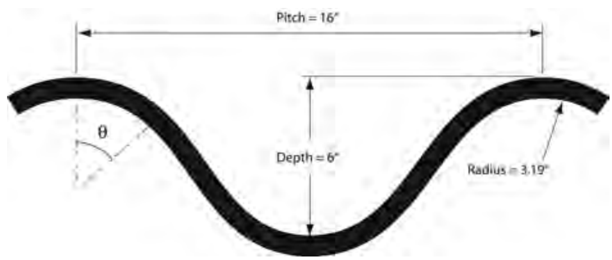


Table 2.15

Sectional properties of 16 x 6 in. (Annular)

Nominal Thickness	Design Thickness	Tangent Length	Tangent Angle	Area of Section	Moment of Inertia	Section Modulus	Plastic Modulus	Radius of Gyration	Developed Width Factor
(in.)	(in.)	(in.)	(Degrees)	($in.^2$/ft)	($in.^4$/in)	($in.^3$/ft)	($in.^3$/ft)	(in.)	
0.169	0.166	4.426	51.294	2.736	0.988	0.311	0.424	2.0813	1.38
0.197	0.195	4.387	51.440	3.218	1.163	0.364	0.498	2.0827	1.38
0.236	0.236	4.331	51.640	3.902	1.413	0.440	0.605	2.0844	1.38
0.276	0.276	4.277	51.840	4.554	1.652	0.511	0.707	2.0863	1.38
0.315	0.313	4.225	52.032	5.166	1.877	0.577	0.803	2.0881	1.38

Notes: 1. Actual pitch = 15.748 in. and actual depth = 5.906 in.
2. Dimensions shown on the sketch have been soft converted on the basis of 1.0 in. = 25 mm.

PIPE SEAMS

The type of pipe seam depends upon both the product and method of manufacture. Most shop-manufactured CSP is produced on a machine that forms helical corrugations; in such case, the seam may be either a continuous helical lock seam or a continuously helical welded seam, depending upon the capabilities of the production facility. However, some shop-manufactured CSP is produced on equipment that forms annular corrugations; in such case, the longitudinal and circumferential seams may be either riveted or spot welded, depending upon the production facilities capabilities and the project specifications. In contrast with CSP, structural plate pipe is always fabricated with annular corrugations and field bolted longitudinal and circumferential seams.

Riveted Seams

Specifications for 2 2/3 x 1/2 inch corrugations call for the use of 5/16 inch diameter rivets for material thickness of 0.064 and 0.079 inches, and 3/8 inch diameter rivets for thickness of 0.109, 0.138, and 0.168 inches. For 3 x 1 inch corrugations, specifications call for 3/8 inch diameter rivets for material thickness of 0.064 and 0.079 inches, and 7/16 inch diameter rivets for thickness of 0.109, 0.138, and 0.168 inches. Longitudinal seams are riveted with one rivet in each corrugation with pipes 42 inches or larger diameter double-riveted. Circumferential rivets for joining sections are spaced on 6 inch centers. The strength of longitudinal seams for steel sheets and rivets is shown in Table 2.16.

Table 2.16

Ultimate longitudinal seam strength of riveted corrugated steel pipe
Tested as uncovered short columns in pounds per foot of seam*

Specified Thickness	5/16 in. Rivets		3/8 in. Rivets			7/16 in. Rivets
	2 2/3 x 1/2 in.		2 2/3 x 1/2 in.		3 x 1 in. and 5 x 1 in.	3 x 1 in. and 5 x 1 in.
(in.)	Single	Double	Single	Double	Double	Double
0.064	16,700	21,600			28,700	
0.079	18,200	29,800			35,700	
0.109			23,400	46,800		53,000
0.138			24,500	49,000		63,700
0.168			25,600	51,300		70,700

Note: Values in this table are based on tests conducted by Utah State Dept. of Highways, 1964, and by Pittsburgh Testing Laboratories, 1966.

Spot Welded Seams

Resistance spot welding of lapped seams is a fabricating method resulting in strength equivalent to riveted seams. Elimination of rivet heads allows a smoother pipe interior and better seating of the connecting band on the exterior.

Bolted Seams and Joints

For structural plate products, high strength bolts, either 3/4 inch or 7/8 inch diameter, hot-dip galvanized, meeting ASTM Specification A 449 are used for field assembly of structural plate installations. Table 2.17 shows the strength of bolted longitudinal seams.

Table 2.17

Ultimate strength of bolted structural plate longitudinal seams
In pounds per foot of seam

Specified Thickness (in.)	6 x 2 in.*			15 x 5 1/2 in. *		16 x 6 in. *	
	4 Bolts per Foot	6 Bolts per Foot	8 Bolts per Foot	4.8 Bolts per Foot	Bolt Diameter (in.)	4.5 Bolts per Foot	Bolt Diameter (in.)
0.111	42000						
0.140	62000			66000	0.75		
0.169						81600	0.75
0.170	81000			87000	0.75		
0.188	93000			102000	0.75		
0.197						118900	0.75
0.218	112000			127000	0.75		
0.236						141300	0.75
0.249	132000			144000	0.75		
0.276						153300	0.75
0.280	144000	180000	194000	144000	0.75		
0.315						153300	0.75
0.249				159000	0.875		
0.276						184000	0.875
0.280				177000	0.875		
0.315						184000	0.875
0.318			235000				
0.380			285000				

Note: * Industry recognized seam strengths for 6 x 2 in. and 15 x 5 1/2 in. are published in ASTM A796. At the time this design manual went to publication, the design seam strengths for 16 x 6 in. were not recognized in ASTM A796. Seam strengths shown for 16 x 6 in. corrugation are proprietary values recommended by the manufacturer.

Table 2.18

Handling weight of corrugated steel pipe (2 2/3 x 1/2 in.)
Estimated average weights - not for specification use*

Inside Diameter (in.)	Specified Thickness (in.)	Approximate Pounds Per Linear Foot**						
		Metallic Coated	Polymer Coated	Full Bituminous Coated	Full Bituminous Coated and Invert Paved	Full Bituminous Coated and Full Paved	Steel Lined	Concrete Lined
12	0.064	10	10	12	15			
	0.079	12	12	14	17			
15	0.064	12	13	15	18	28		
	0.079	15	16	18	21	31		
18	0.064	15	16	19	22	34		
	0.079	18	19	22	25	37		
21	0.064	17	18	21	26	39		
	0.079	21	22	25	30	43		

Notes: Pipe arch weights will be the same as the equivalent round pipe.
For example, for 42 x 29, 2 2/3 x 1/2 in. pipe arch, refer to 36 in. diameter pipe weight.
* Lock seam construction only; weights will vary with other fabrication practices.
** For other coatings or linings, the weights may be interpolated.

Table 2.18 *continued*

Handling weight of corrugated steel pipe (2 2/3 x 1/2 in.)
Estimated average weights - not for specification use*

Inside Diameter (in.)	Specified Thickness (in.)	Approximate Pounds Per Linear Foot**						
		Metallic Coated	Polymer Coated	Full Bituminous Coated	Full Bituminous Coated and Invert Paved	Full Bituminous Coated and Full Paved	Steel Lined	Concrete Lined
24	0.064	19	20	24	30	45	30	65
	0.079	24	25	29	35	50	38	69
	0.109	33	34	38	44	59	47	77
30	0.064	24	25	30	36	55	42	82
	0.079	30	31	36	42	60	48	87
	0.109	41	42	47	53	72	59	96
36	0.064	29	30	36	44	65	51	98
	0.079	36	37	43	51	75	58	104
	0.109	49	50	56	64	90	71	116
	0.138	62	63	69	77	100	84	127
42	0.064	34	36	42	51	77	60	114
	0.079	42	44	50	59	85	68	121
	0.109	57	59	65	74	105	82	135
	0.138	72	74	80	89	115	98	149
48	0.064	38	40	48	57	85	67	128
	0.079	48	50	58	67	95	77	138
	0.109	65	67	75	84	120	94	154
	0.138	82	84	92	101	130	111	170
	0.168	100		110	119	155		186
54	0.079	54	56	65	76	105	87	156
	0.109	73	75	84	95	130	106	173
	0.138	92	94	103	144	155	125	191
	0.168	112		123	134	175		
60	0.109	81	83	92	106	140	117	192
	0.138	103	105	114	128	180	139	212
	0.168	124		135	149	190		232
66	0.109	89	91	101	117	160	129	211
	0.138	113	115	125	141	180	153	233
	0.168	137		149	165	210		255
72	0.138	123	126	137	154	210	167	254
	0.168	149		163	180	236		278
78	0.168	161		177	194	260		302
84	0.168	173		190	208	270		325

Notes: Pipe arch weights will be the same as the equivalent round pipe.
For example, for 42 x 29, 2 2/3 x 1/2 in. pipe arch, refer to 36 in. diameter pipe weight.
* Lock seam construction only; weights will vary with other fabrication practices.
** For other coatings or linings, the weights may be interpolated.

Table 2.19

Handling weight of corrugated steel pipe (3 x 1 in. or 5 x 1* in.)
Estimated average weights – not for specification use**

Inside Diameter (in.)	Specified Thickness (in.)	Approximate Pounds Per Linear Foot***						
		Metallic Coated	Polymer Coated	Full Bituminous Coated	Full Bituminous Coated and Invert Paved	Full Bituminous Coated and Full Paved	Steel Lined†	Concrete Lined
48	0.064	44	46	54	71	117	74	
	0.079	54	56	64	81	127	84	
	0.109	74	76	84	101	147	104	
	0.138	94	96	104	121	167	125	
	0.168	114		124	141	187		
54	0.064	50	52	66	84	138	84	197
	0.079	61	63	77	95	149	95	207
	0.109	83	85	100	118	171	118	226
	0.138	106	108	123	140	194	140	245
	0.168	129		146	163	217		264
60	0.064	55	57	73	93	153	93	218
	0.079	67	69	86	105	165	105	229
	0.109	92	94	110	130	190	130	251
	0.138	118	120	136	156	216	156	272
	0.168	143		161	181	241		293
66	0.064	60	63	80	102	168	102	240
	0.079	74	77	94	116	181	116	252
	0.109	101	104	121	143	208	145	276
	0.138	129	132	149	171	236	172	299
	0.168	157		177	199	264		322
72	0.064	66	69	88	111	183	112	262
	0.079	81	84	102	126	197	127	275
	0.109	110	113	132	156	227	157	301
	0.138	140	143	162	186	257	187	326
	0.168	171		193	217	288		351
78	0.064	71	74	95	121	198	120	
	0.079	87	90	111	137	214	136	298
	0.109	119	122	143	169	246	168	326
	0.138	152	155	176	202	279	202	353
	0.168	185		209	235	312		380
84	0.064	77	80	102	130	213	130	
	0.079	94	97	119	147	230	147	321
	0.109	128	131	154	182	264	181	351
	0.138	164	167	189	217	300	218	379
	0.168	199		224	253	335		409
90	0.064	82	86	109	140	228	139	
	0.079	100	104	127	158	246	157	
	0.109	137	141	164	195	283	194	376
	0.138	175	179	202	233	321	233	406
	0.168	213		240	271	359		438

Notes: Pipe arch weights will be the same as the equivalent round pipe. For example: for 81 x 59, 3 x 1 in. pipe arch, refer to 72 in. diameter pipe weight.
* Steel weights are 5 x 1 in. are approximately 12% less than those used in this table for metallic coated pipe.
** Lock seam construction only, weights will vary with other fabrication practices.
*** For other coatings or linings the weights may be interpolated.
† Steel lined available in 3 x 1 in. only.

Table 2.19 *continued*

Handling weight of corrugated steel pipe (3 x 1 in. or 5 x 1* in.)
Estimated average weights – not for specification use**

Inside Diameter (in.)	Specified Thickness (in.)	Approximate Pounds Per Linear Foot***						
		Metallic Coated	Polymer Coated	Full Bituminous Coated	Full Bituminous Coated and Invert Paved	Full Bituminous Coated and Full Paved	Steel Lined†	Concrete Lined
96	0.064	87	91	116	149	242	148	
	0.079	107	111	136	169	262	168	
	0.109	147	151	176	209	302	208	401
	0.138	188	192	217	250	343	249	433
	0.168	228		257	290	383		467
102	0.064	93	97	124	158	258	158	
	0.079	114	118	145	179	279	179	
	0.109	155	159	189	220	320	222	426
	0.138	198	202	229	263	363	264	460
	0.168	241		272	306	406		496
108	0.079	120	124	153	188	295	189	
	0.109	165	169	198	233	340	235	
	0.138	211	215	244	279	386	279	487
	0.168	256		289	324	431		525
114	0.079	127	132	162	199	312	200	
	0.109	174	179	209	246	359	248	
	0.138	222	227	257	294	407	295	514
	0.168	271		306	343	456		554
120	0.109	183	188	220	259	378	260	
	0.138	234	239	271	310	429	311	541
	0.168	284		321	360	479		583
126	0.109	195	200	233	274	400	276	
	0.138	247	252	285	326	452	327	
	0.168	299		337	378	504		
132	0.109	204	209	244	287	419	289	
	0.138	259	264	299	342	474	343	
	0.168	314		354	397	529		
138	0.109	213	219	255	300	438	300	
	0.138	270	276	312	357	495	357	
	0.168	328		370	415	553		
144	0.138	282	288	326	373	517	373	
	0.168	344		388	435	579		
150	0.138	294	300	340	389	538	389	
	0.168	358		404	453	602		
156	0.138	306	312	354	406	560	405	
	0.168	373		421	473	627		

Notes: Pipe arch weights will be the same as the equivalent round pipe. For example: for 81 x 59, 3 x 1 in. pipe arch, refer to 72 in. diameter pipe weight.
* Steel weights are 5 x 1 in. are approximately 12% less than those used in this table for metallic coated pipe.
** Lock seam construction only, weights will vary with other fabrication practices.
*** For other coatings or linings the weights may be interpolated.
† Steel lined available in 3 x 1 in. only.

Table 2.20

Handling weight of spiral rib pipe and composite ribbed steel pipe
(3/4 x 3/4 x 71/2 in & 3/4 x 1 x 111/2 in. spiral rib pipe and 3/4 x 3/4 x 71/2 in. composite ribbed steel pipe)
Estimated average weights – not for specification use*

Inside Diameter (in.)	Specified Thickness (in.)	Approximate Pounds Per Linear Foot**				Composite Ribbed Steel Pipe
		Galvanized	Asphalt Fully Coated	Asphalt Fully Coated & Invert Paved		
18	0.064	15	19	20		
	0.079	18	22	23		
21	0.064	17	21	22		
	0.079	21	25	26		
	0.109	29	33	33		
24	0.064	19	24	25		21
	0.079	24	29	32		25
	0.109	36	41	42		33
30	0.064	24	30	32		27
	0.079	30	36	38		32
	0.109	42	48	50		41
36	0.064	29	36	38		32
	0.079	36	43	45		28
	0.109	50	57	59		49
42	0.064	33	41	43		37
	0.079	42	50	52		44
	0.109	58	66	60		57
48	0.064	38	48	50		43
	0.079	48	58	60		50
	0.109	66	76	78		66
54	0.064	43	54	56		48
	0.079	54	65	67		56
	0.109	75	86	88		74
60	0.064	48	60	62		53
	0.079	60	72	74		62
	0.109	83	95	97		82
	0.138	99+	111 +			
66	0.064***	53	66	68		58
	0.079	66	79	81		69
	0.109	91	104	106		90
	0.138	109+	121 +			
72	0.064					63
	0.079	72	86	89		75
	0.109	99	113	116		98
	0.138	119+	133 +			
78	0.079	78	93	96		81
	0.109	108	115	118		106
	0.138	129+	144 +			
84	0.079 ***	71	101	104		87
	0.109	116	133	136		114
	0.138	139+	156 +			
90	0.109	124	143	147		122
	0.138	149+	168 +			
96	0.109	132	152	156		130
	0.138	158+	178 +			
102	0.109	141	163	167		138
	0.138	168+	190 +			
108	0.109 ***	150	172	176		146
	0.138	175+	197 +			
114	0.138	196	219	223		
120	0.138	206	230	235		

Notes: * Lock seam construction only. ** For other coatings or linings, the weights may be interpolated.
*** For 3/4 x 1 x 11 1/2 in. only. + For 3/4 x 3/4 x 7 1/2 in. only.

Relining of a failed concrete box with corrugated steel pipe arch.

SPIRAL RIB STEEL PIPE

Spiral rib pipe is manufactured from a continuous strip of metallic coated or polymer coated steel passed through a roll forming line that forms the external ribs and the edges. The rolled shape section is then helically formed into pipe and the edges are joined by lock seaming. The finished product has the structural characteristics needed for installation and a smooth interior for improved hydraulics. See Tables 2.8 through 2.12 for profile shapes.

Spiral rib pipe.

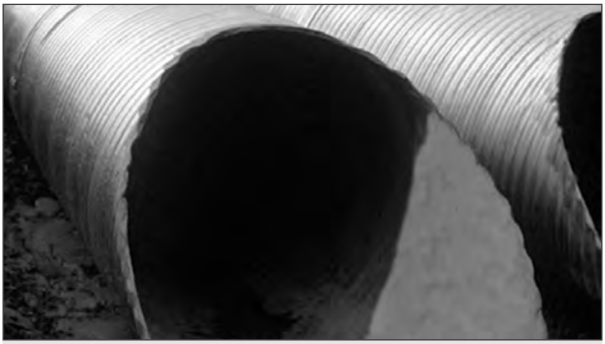
DOUBLE WALL STEEL PIPE

Double Wall (steel lined) is a smooth interior corrugated steel pipe fabricated in full circular cross section with a smooth steel liner and helically corrugated shell integrally attached at the helical lock seams from end to end of each length of pipe. The steel interior lining provides for improved hydraulics.

Double wall pipe.

CSP CONCRETE LINED PIPE

The interior lining of the corrugated steel pipe is composed of an extremely dense, high strength concrete. The lining provides a superior wearing surface for extended structure life as well as a smooth interior for improved hydraulics.



Concrete lined corrugated steel pipe.

COMPOSITE RIBBED STEEL PIPE

Composite ribbed steel pipe is manufactured from a continuous strip of metallic coated steel passed through a roll forming mill that forms external ribs. The coated steel is protected by a polymer film on the outside and has a 75 mil polyethylene interior liner for protection from effluent corrosion and/or abrasion as well as providing a smooth interior for improved hydraulics.

CSP SLOTTED DRAIN INLETS

By welding a narrow section of grating in a continuous slot cut in the top of a corrugated steel pipe, a continuous grate inlet is achieved. Originally conceived to pick up sheet flow in roadway medians, parking lots, airports, etc., this product has proven even more useful as a curb inlet. Detailed hydraulic design information is provided in Chapter 4.

■ Slotted drain inlet pipe.

■ Slotted drain inlet pipe.

■ Slotted drain inlet pipe.

■ Slotted drain inlet pipe.

PERFORATED PIPE

Corrugated steel pipe is available with perforations for collection or dissemination of water underground and is an effective means of storm water management. Subsurface, or groundwater control, is the most common use for perforated corrugated steel pipe. In this application, only the lower half of the pipe is perforated as shown in Table 2.21. Most fabricators are equipped to furnish 3/8 inch diameter holes. The sizes and layout of perforations can be specified to match site requirements. The perforations are located on the inside crests or along the neutral axis of the corrugations, with one row of perforations in each corrugation.

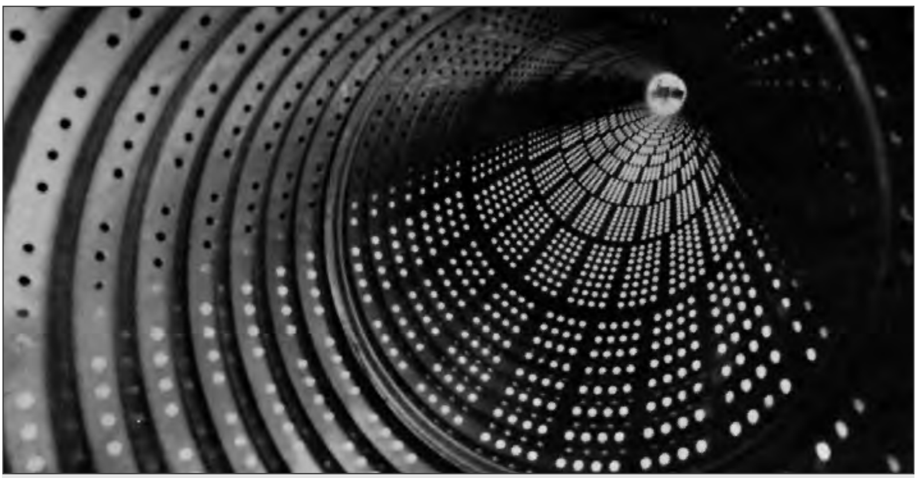
Fully perforated helical CSP is ideally suited for retention of storm water, permitting slow infiltration, or recharge, into the trench walls. Underground disposal of storm water runoff in urban development design has the potential for saving millions of dollars in taxpayer money. Recharge design makes the concept of zero increase in runoff possible thus avoiding overloading trunk storm drains, and/or streams and rivers. The cost of reconstructing existing drains or channel improvements will usually prove to be far greater than recharge design. In the retention application, the pipe is typically perforated for the full 360 degrees. Perforations in fully perforated helical pipe usually provide an opening area of not less than 2.3% of the pipe surface.

Table 2.21

Perforated galvanized corrugated pipe data*
Dimensions, Weights, Perforations

Nominal Perforations		Minimum Width of Unperforated Bottom Segment	Specified Thickness			
			0.052	0.064	0.052	0.064
			Weight, lbs per ft			
Internal Diameter	Number of Rows		Helically Corrugated Pipe		Annular Corrugated Pipe	
(in.)	(in.)	(in.)	(in.)	(in.)	(in.)	(in.)
6	4	3.8	3.8	4.7	5.0	5.6
8	4	5.1	5.0	6.2	6.3	7.3
10	4	6.4	6.5	7.6	-	9.0
12	6	7.7	-	9.9	-	10.5
15	6	9.6	-	12.4	-	12.9
18	6	11.5	-	14.8	-	15.3
21	6	13.5	-	17.2	-	17.7
24	8	15.4	-	19.3	-	20.0

Note: * AASHTO Spec. M 36.



The pipe is perforated for the full 360° to be used in a subsurface recharge system.

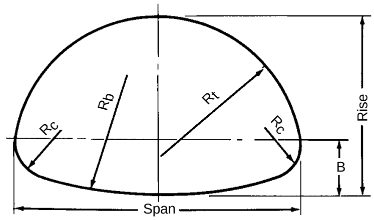


Table 2.22

Sizes and layout details — CSP pipe arches
(2 2/3 x 1/2 in. corrugation)

Equiv. Diameter	Design Span	Design Rise	Waterway Area	Layout Dimensions B	Layout Dimensions R_c	Layout Dimensions R_t	Layout Dimensions R_b
(in.)	(in.)	(in.)	(ft^2)	(in.)	(in.)	(in.)	(in.)
15	17	13	1.1	4 1/8	3 1/2	8 5/8	25 5/8
18	21	15	1.6	4 7/8	4 1/8	10 3/4	33 1/8
21	24	18	2.2	5 5/8	4 7/8	11 7/8	34 5/8
24	28	20	2.9	6 1/2	5 1/2	14	42 1/4
30	35	24	4.5	8 1/8	6 7/8	17 7/8	55 1/8
36	42	29	6.5	9 3/4	8 1/4	21 1/2	66 1/8
42	49	33	8.9	11 3/8	9 5/8	25 1/8	77 1/4
48	57	38	11.6	13	11	28 5/8	88 1/4
54	64	43	14.7	14 5/8	12 3/8	32 1/4	99 1/4
60	71	47	18.1	16 1/4	13 3/4	35 3/4	110 1/4
66	77	52	21.9	17 7/8	15 1/8	39 3/8	121 1/4
72	83	57	26.0	19 1/2	16 1/2	43	132 1/4

Note: Layout dimensions are typical manufactured dimensions. Specified dimensions are found in ASTM A760.

Table 2.23

Sizes and layout details — CSP pipe arches
(3 x 1 or 5 x 1 in. corrugation)

Equiv. Diameter	Nominal Size	Design Span	Design Rise	Waterway Area	Layout Dimensions B	Layout Dimensions R_c	Layout Dimensions R_t	Layout Dimensions R_b
(in.)	(in.)	(in.)	(in.)	(ft^2)	(in.)	(in.)	(in.)	(in.)
48	53 x 41	53	41	11.7	15 1/4	10 3/16	28 1/16	73 7/16
54	60 x 46	58 1/2	48 1/2	15.6	20 1/2	18 3/4	29 3/8	51 1/8
60	66 x 51	65	54	19.3	22 3/4	20 3/4	32 5/8	56 1/4
66	73 x 55	72 1/2	58 1/4	23.2	25 1/8	22 7/8	36 3/4	63 3/4
72	81 x 59	79	62 1/2	27.4	23 3/4	20 7/8	39 1/2	82 5/8
78	87 x 63	86 1/2	67 1/4	32.1	25 3/4	22 5/8	43 3/8	92 1/4
84	95 x 67	93 1/2	71 3/4	37.0	27 3/4	24 3/8	47	100 1/4
90	103 x 71	101 1/2	76	42.4	29 3/4	26 1/8	51 1/4	111 5/8
96	112 x 75	108 1/2	80 1/2	48.0	31 5/8	27 3/4	54 7/8	120 1/4
102	117 x 79	116 1/2	84 3/4	54.2	33 5/8	29 1/2	59 3/8	131 3/4
108	128 x 83	123 1/2	89 1/4	60.5	35 5/8	31 1/4	63 1/4	139 3/4
114	137 x 87	131	93 3/4	67.4	37 5/8	33	67 3/8	149 1/2
120	142 x 91	138 1/2	98	74.5	39 1/2	34 3/4	71 5/8	162 3/8
126	150 x 96	146	102	81	41	36	76	172
132	157 x 101	153	107	89	43	38	80	180
138	164 x 105	159	113	98	45	40	82	184
144	171 x 110	165	118 1/2	107	47	41	85	190

Note: Layout dimensions are typical manufactured dimensions. Specified dimensions are found in ASTM A760.

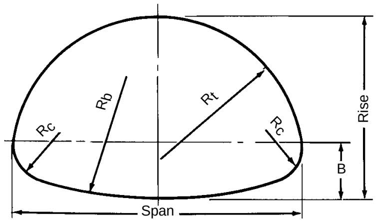


Table 2.24

Sizes and layout details, spiral rib pipe arch and composite ribbed steel pipe arch

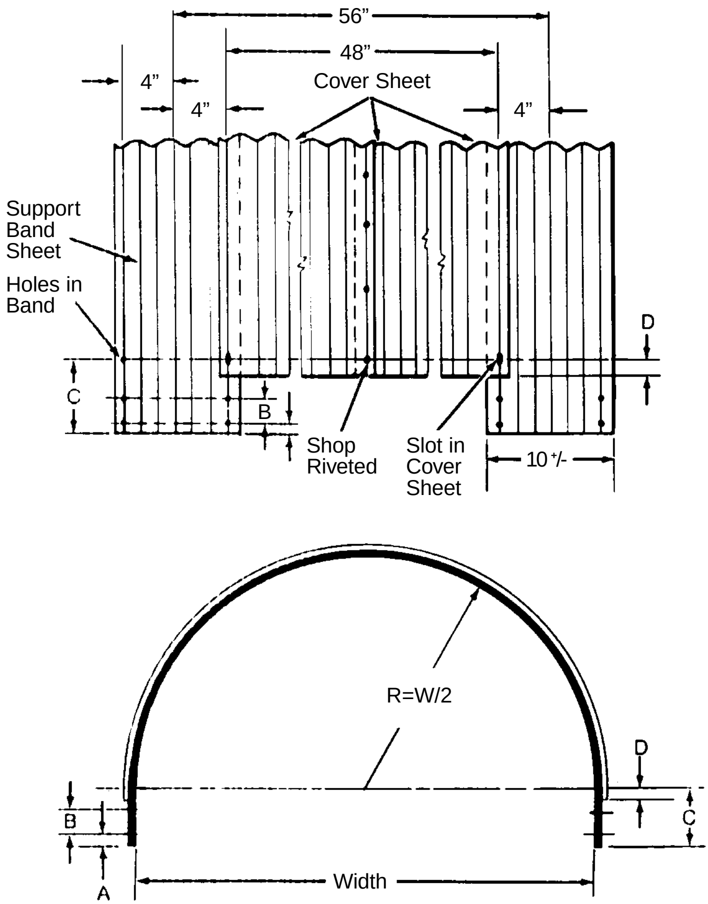
Equiv. Diameter	Design		Waterway Area	Layout Dimensions			
	Span	Rise		B	R_c	R_t	R_b
(in.)	(in.)	(in.)	(ft^2)	(in.)	(in.)	(in.)	(in.)
18	20	16	1.7	5 1/8	5	10 1/4	27 1/2
21	23	19	2.3	5 7/8	5 3/8	11 5/8	34 1/4
24	27	21	3.0	6 3/4	5 3/4	13 1/2	40 7/8
30	33	26	4.7	8 3/4	7 1/8	16 5/8	51 3/8
36	40	31	6.7	10 3/8	8 3/8	20 1/4	62 1/2
42	46	36	9.2	12 3/8	9 3/4	23 1/4	73
48	53	41	12.1	14	11 1/8	26 5/8	83 1/2
54	60	46	15.6	20 1/2	18 3/4	29 3/8	51 1/8
60	66	51	19.3	22 3/4	20 3/4	32 5/8	56 1/4
66	73	55	23.2	25 1/8	22 7/8	36 3/4	63 3/4
72	81	59	27.4	23 3/4	20 7/8	39 1/2	82 5/8
78	87	63	32.1	25 3/4	22 5/8	43 3/8	92 1/4
84	95	67	37.0	27 3/4	24 3/8	47	100 1/4
90	103	71	42.4	29 3/4	26 1/8	51 1/4	111 5/8
96	112	75	48.0	31 5/8	27 3/4	54 7/8	120 1/4
102	117	79	54.2	33 5/8	29 1/2	59 3/8	131 3/4

Notes: Layout dimensions are typical manufactured dimensions. Specified dimensions are found in ASTM A760.

CONVEYOR COVERS

Arch Sections. Perhaps the most commonly used cover is a half-circle steel arch section, 48 inches long, supported on band sheets 10 inches wide. See Figure 2.3. These band sheets in turn are supported by bolting to the conveyor frame. Diameters of support bands and cover sheets are optional, to meet the conveyor equipment manufacturer's designs, but usually range from 36 to 72 inches, in suitable thicknesses of steel. Cover sheets are secured by one bolt at each corner and can be removed quickly when necessary. Corrugations should run transverse to the conveyor for greater strength with minimum framing. Where the arch covers not only the conveyor belt, but also the walkway, sheets with larger corrugations (6 x 2 inches) can be provided.

Horseshoe or Full Round. The horseshoe shape finds use where weighing equipment or other facilities require a larger cover. A circular or elliptical shape can also serve as a beam to strengthen the span between bents.

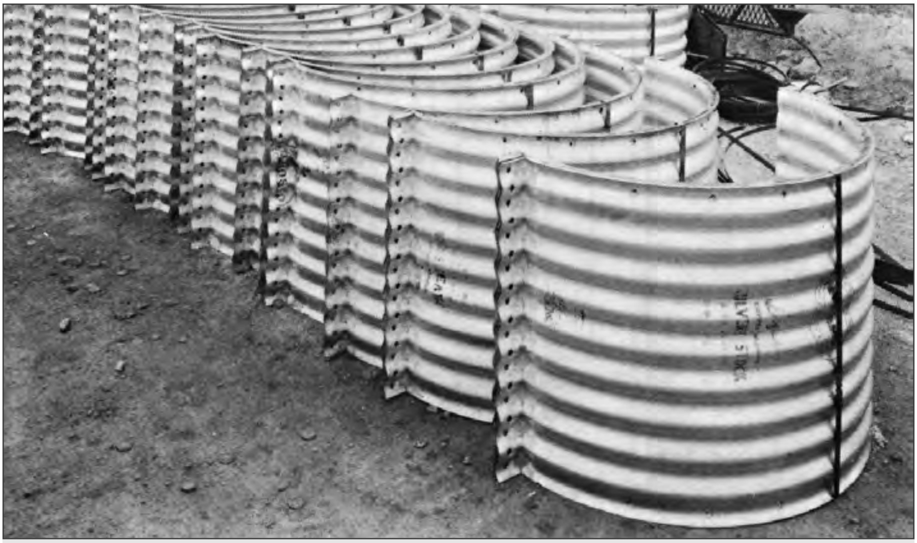


■ **Figure 2.3** Typical corrugated steel conveyor cover - with removable cover sheets supported by narrower band arches.

NESTABLE CORRUGATED STEEL PIPE

Nestable pipe offers a fast and economical solution to contractors and owners who require a strong casing to place around an already installed utility line. This can be done easily without disrupting the line to be encased.

There are two standard methods used in attaching the half-round pipe segments together, interlocking notches and mating flanges. Nesting, a shipping technique developed in the 1930’s, was devised to eliminate problems for overseas shipment. It provides an economical solution to conserve shipping space.



Nestable pipe segments to be assembled into corrugated steel pipe.

STRUCTURAL PLATE PRODUCTS

Product Description

Structural plate pipes are structures where corrugated steel sections are bolted together to form the required shape. The corrugated sections are commonly referred to as plates. The 6 x 2 inch corrugation shown in Figure 2.1 is the standard. Structural plate structures are specified where the pipe required exceeds the size that can be shipped to the job site, or where earth cover is so great that the wall thickness furnished by a shop-manufactured pipe will not meet design requirements.

The corrugations are formed at right angles to the length of the bridge or culvert. The length of a plate is measured in a direction parallel to the length of the structure. The width of a plate is, therefore, measured in a direction perpendicular to the length of the structure, around the periphery of the structure. See Figures 2.4 and 2.5.

Standard plates are fabricated in three lengths and several different widths. The plate width designation, N, is used to describe the various plate widths available. N is the spacing between two circumferential bolts, or one circumferential bolt hole space (circumferential refers to the direction around the periphery of the structure, at right angles to the length of the structure). For instance, a 5N plate has a net width of 5 circumferential bolt hole spaces and an 8N plate has a net width of 8 circumferential bolt hole spaces. The bolt hole space, N, is 9.6 inches (see Table 2.25). Note that not all widths are available in all lengths.

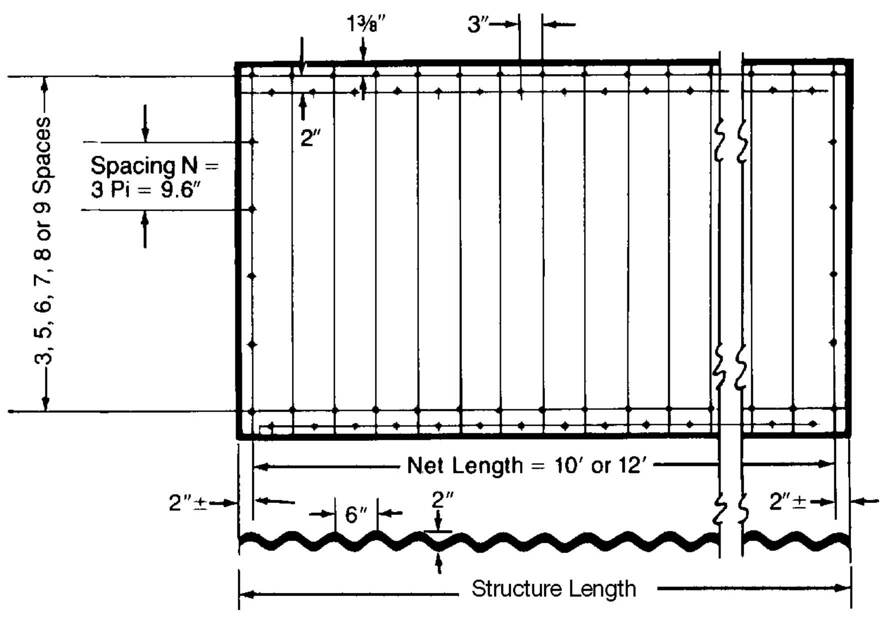
Plates are furnished curved to various radii and are clearly identified by the fabricator for field assembly. The fabricator provides assembly drawings to guide the installer. The plates are available in thicknesses from 0.111 inches to 0.380 inches See Table 2.13 for sectional properties. Weights of individual plate sections are shown in Table 2. 26. Approximate weights of structural plate structures are readily calculated using these values.

Section Properties

Section properties, used for design, are provided in Table 2.13. As with corrugated steel pipe corrugations, properties of the arc-and-tangent structural plate corrugation are derived mathematically using the design thickness. The properties in the table include area, moment of inertia, section modulus and radius of gyration.

Sizes and Shapes

The plates are assembled into various shapes as indicated in Tables 2.27 through 2.36. The shapes include round, pipe arch, single-radius arch, horizontal ellipse, low profile arch, high profile arch, pear, underpass and vertical ellipse. Special shapes, and other sizes of standard shapes beyond what is shown in the tables, are also available. The fabricator provides detailed assembly instructions with each structure.



■ **Figure 2.4** Details of 6 x 2 in. uncurved structural plate from a domestic source.

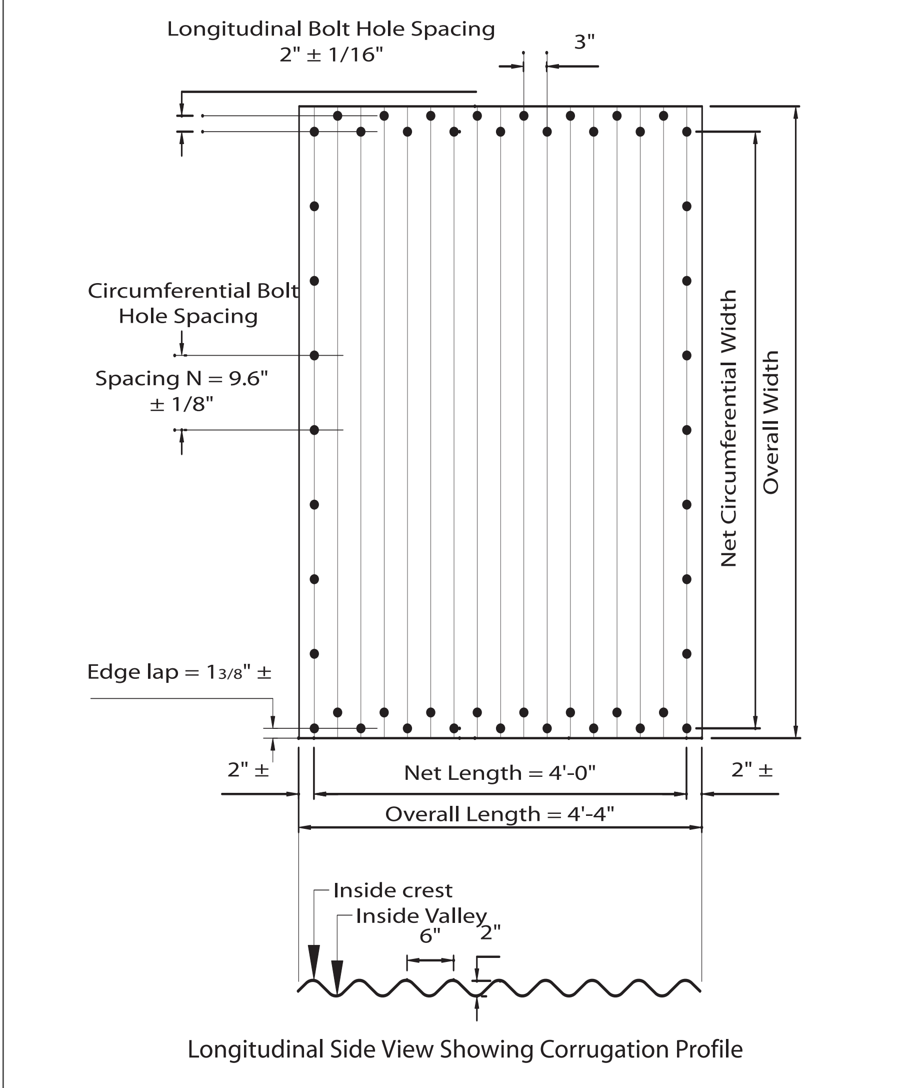


Figure 2.5 Non-domestic 6 x 2 in. structural plate configuration.

Table 2.25

6 x 2 in. corrugated structural plate sections — details of uncurved plates

Nominal Plate Width, N*	Net Width		Overall Width	Number of Circumference Bolt Holes
	(in.)		(in.)	
3N	28.8	28 13/16	33 9/16	4
4N	38.4	38 3/8	43 1/8	5
5N	48.0	48	52 3/4	6
6N	57.6	57 5/8	62 3/8	7
7N	67.2	67 3/16	71 15/16	8
8N	76.8	76 13/16	81 9/16	9
9N	86.4	86 3/8	91 1/8	10
10N	96.0	96	100 3/4	11
11N	105.6	105 5/8	110 3/8	12
12N	115.2	115 3/16	119 15/16	13
13N	124.8	124 13/16	129 9/16	14
14N	134.4	134 3/8	139 1/8	15
15N	144.0	144	148 3/4	16
16N	153.6	153 5/8	158 3/8	17

Note: *N = 3 Pi = 9.6 inches

Nested corrugated steel pipe.

Table 2.26

Weight of 6 x 2 in. corrugated structural plate sections

Nominal Plate Width, N*	Net Length ft	Approx. Wt. Of Individual Galvanized Plates Without Bolts, in Pounds Specified Thickness, in.									Number of Assembly Bolts/Plate
		0.111	0.140	0.170	0.188	0.218	0.249	0.280	0.318	0.380	
3N	10	161	205	250	272	316	361	405	468	559	42
3N	12	193	246	299	325	379	432	485	560	669	50
5N	10	253	323	393	428	498	568	638	730	872	44
5N	12	303	386	470	511	595	678	762	873	1043	52
6N	10	299	382	465	506	589	671	754	859	1026	45
6N	12	357	456	555	604	703	801	900	1027	1227	53
7N	10	345	441	536	583	679	774	869	982	1185	46
7N	12	412	526	640	697	810	924	1038	1186	1417	54
8N	10	396	504	613	667	775	878	986			47
8N	12	473	603	732	797	927	1050	1176			55
9N	10	431	548	666	743	865	986	1108			48
9N	12	517	657	799	892	1038	1183	1330			56
3N	4	72	91	111	122	142	162	182			18
4N	4	93	117	142	157	182	208	234			19
5N	4	113	143	174	192	222	254	286			20
6N	4	134	169	205	227	263	300	338			21
7N	4	154	195	236	261	303	346	389			22
8N	4	175	221	268	296	343	392	441			23
9N	4	195	246	299	331	384	438	493			24
10N	4	216	272	331	366	424	484	545			25
11N	4	236	298	362	400	464	530	596			26
12N	4	257	324	394	435	505	576	648			27
13N	4	277	350	425	470	545	622	670			28
14N	4	298	376	456	505	585	668	752			29
15N	4	318	402	488	539	626	714	803			30
16N	4	339	428	519	574	666	760	855			31

Notes: 1 Weights are approximate, based on standard 2 holes per corrugation in longitudinal seams. Plates are galvanized with an average coating mass of 3 oz/ft^2 (total both surfaces).

2 For galvanized plate thicknesses 0.111 to 0.188 in., bolt lengths are 1 1/4 and 1 1/2 in.; for thicknesses 0.218 and 0.249 in., bolt lengths are 1 1/2 and 1 3/4 in.; for thickness 0.280 in. bolt lengths are 1 1/2 and 2 in. Bolts are color coded for the different lengths.

3 Weight of bolts and nuts in lbs. per hundred:
1 1/4 in. = 52 2 in. = 59.5
1 1/2 in. = 55 3 in. = 72.5
1 3/4 in. = 57

4 To compute the approx. wt. of structures per foot of length: (1) multiply the number of plates in the periphery by the plate weights from the table; (2) add weight of the bolts; (3) divide by plate length.

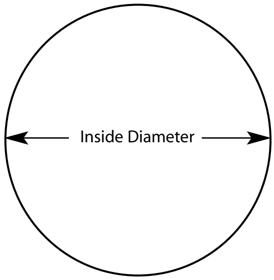


Table 2.27

Structural plate pipe — sizes and end areas

Pipe Diameter (ft)	End Area (ft^2)	Periphery No. of Plates	N	Pipe Diameter (ft)	End Area (ft^2)	Periphery No. of Plates	N
5.0	20	4	20	16.0	201	10	64
5.5	24	4	22	16.5	214	10	66
6.0	28	4	24	17.0	227	10	68
6.5	33	4	26	17.5	241	10	70
7.0	38	4	28	18.0	254	12	72
7.5	44	6	30	18.5	269	12	74
8.0	50	6	32	19.0	284	12	76
8.5	57	6	34	19.5	299	12	78
9.0	64	6	36	20.0	314	12	80
9.5	71	6	38	20.5	330	12	82
10.0	79	6	40	21.0	346	12	84
10.5	87	6	42	21.5	363	14	86
11.0	95	8	44	22.0	380	14	88
11.5	104	8	46	22.5	398	14	90
12.0	113	8	48	23.0	415	14	92
12.5	123	8	50	23.5	434	14	94
13.0	133	8	52	24.0	452	14	96
13.5	143	8	54	24.5	470	14	98
14.0	154	8	56	25.0	491	16	100
14.5	165	10	58	25.5	510	16	102
15.0	177	10	60	26.0	530	16	104
15.5	189	10	62				

Fish Passage Project on the Mc Cloud River under the Mc Cloud Rail Road in Northern California. 16 foot and 20 foot diameter structural plate pipes.

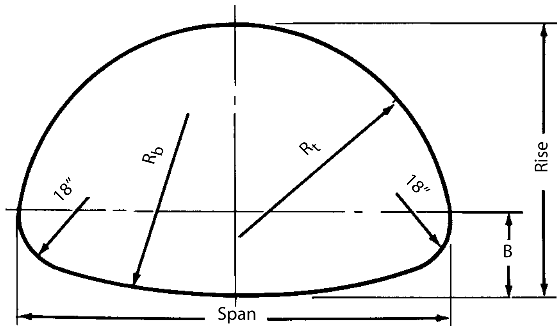


Table 2.28

Structural plate pipe arch size and layout details
6 x 2 in. corrugation — bolted seams 18 in. corner radius, R_c

Dimensions		Waterway	Layout Dimensions			Periphery	
Span (ft-in.)	Rise (ft-in.)	Area (ft^2)	B (in.)	R_t (ft)	R_b (ft)	No. of Plates	Total N
6-1	4-7	22	21.0	3.07	6.36	5	22
6-4	4-9	24	20.5	3.18	8.22	5	23
6-9	4-11	26	22.0	3.42	6.96	5	24
7-0	5-1	28	21.4	3.53	8.68	5	25
7-3	5-3	31	20.8	3.63	11.35	6	26
7-8	5-5	33	22.4	3.88	9.15	6	27
7-11	5-7	35	21.7	3.98	11.49	6	28
8-2	5-9	38	20.9	4.08	15.24	6	29
8-7	5-11	40	22.7	4.33	11.75	7	30
8-10	6-1	43	21.8	4.42	14.89	7	31
9-4	6-3	46	23.8	4.68	12.05	7	32
9-6	6-5	49	22.9	4.78	14.79	7	33
9-9	6-7	52	21.9	4.86	18.98	7	34
10-3	6-9	55	23.9	5.13	14.86	7	35
10-8	6-11	58	26.1	5.41	12.77	7	36
10-11	7-1	61	25.1	5.49	15.03	7	37
11-5	7-3	64	27.4	5.78	13.16	7	38
11-7	7-5	67	26.3	5.85	15.27	8	39
11-10	7-7	71	25.2	5.93	18.03	8	40
12-4	7-9	74	27.5	6.23	15.54	8	41
12-6	7-11	78	26.4	6.29	18.07	8	42
12-8	8-1	81	25.2	6.37	21.45	8	43
12-10	8-4	85	24.0	6.44	26.23	8	44
For sizes below, consider using pipe arch with 31 in. corner radius if cover limits permit. (See Table 2.29)							
13-5	8-5	89	26.3	6.73	21.23	9	45
13-11	8-7	93	28.9	7.03	18.39	9	46
14-1	8-9	97	27.6	7.09	21.18	9	47
14-3	8-11	101	26.3	7.16	24.80	9	48
14-10	9-1	105	28.9	7.47	21.19	9	49
15-4	9-3	109	31.6	7.78	18.90	9	50
15-6	9-5	113	30.2	7.83	21.31	10	51
15-8	9-7	118	28.8	7.89	24.29	10	52
15-10	9-10	122	27.4	7.96	28.18	10	53
16-5	9-11	126	30.1	8.27	24.24	10	54
16-7	10-1	131	28.7	8.33	27.73	10	55

Notes: Dimensions are to inside crests and are subject to manufacturing tolerances. N = 3 Pi = 9.6 in.

Pipe arches larger than 12'-10" x 8'-4" should be specified with a 31" corner radius unless the application is an extension of an existing pipe arch with an 18" corner radius or the relining of an existing culvert. For other applications involving larger size pipe arches with 18" corner radii, consult with the structural plate manufacturer.

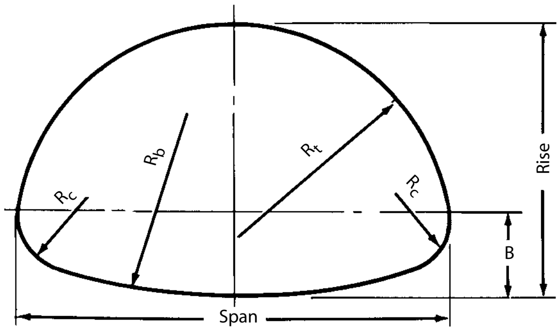


Table 2.29

Structural plate pipe arch — size and layout details
6 x 2 in. corrugation — bolted seams – 31 in. corner radius, R_C

Dimensions		Waterway Area (ft^2)	Layout Dimensions			Periphery	
Span (ft-in.)	Rise (ft-in.)		B (in.)	R_t (ft)	R_b (ft)	No. of Plates	Total N
13-3	9-4	97	38.5	6.68	16.05	8	46
13-6	9-6	102	37.7	6.78	18.33	8	47
14-0	9-8	105	39.6	7.03	16.49	8	48
14-2	9-10	109	38.8	7.13	18.55	8	49
14-5	10-0	114	37.9	7.22	21.38	8	50
14-11	10-2	118	39.8	7.48	18.98	9	51
15-4	10-4	123	41.8	7.76	17.38	9	52
15-7	10-6	127	40.9	7.84	19.34	10	53
15-10	10-8	132	40.0	7.93	21.72	10	54
16-3	10-10	137	42.1	8.21	19.67	10	55
16-6	11-0	142	41.1	8.29	21.93	10	56
17-0	11-2	146	43.3	8.58	20.08	10	57
17-2	11-4	151	42.3	8.65	22.23	10	58
17-5	11-6	157	41.3	8.73	24.83	10	59
17-11	11-8	161	43.5	9.02	22.55	10	60
18-1	11-10	167	42.4	9.09	24.98	10	61
18-7	12-0	172	44.7	9.38	22.88	10	62
18-9	12-2	177	43.6	9.46	25.19	10	63
19-3	12-4	182	45.9	9.75	23.22	10	64
19-6	12-6	188	44.8	9.83	25.43	11	65
19-8	12-8	194	43.7	9.90	28.04	11	66
19-11	12-10	200	42.5	9.98	31.19	11	67
20-5	13-0	205	44.9	10.27	28.18	11	68
20-7	13-2	211	43.7	10.33	31.13	12	69

Note: Dimensions are to inside crests and are subject to manufacturing tolerances. N = 3 Pi = 9.6 in.

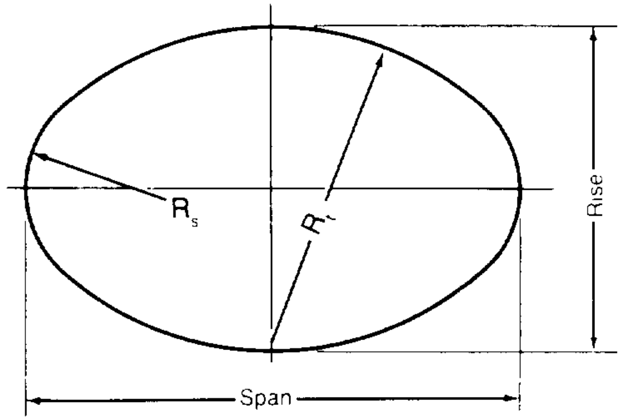


Table 2.30

Structural plate horizontal ellipse — size and layout details
6 x 2 in. corrugation — bolted seams

Span (ft-in.)	Rise (ft-in.)	Area (ft^2)	Periphery: Top or Bottom N	Periphery: Side N	Periphery: Total N	Inside Radius: R_t Top Radius (ft)	Inside Radius: R_s Side Radius (ft)
7-4	5-6	31.3	8	5	26	4-6	2-2
8-1	5-9	36.4	9	5	28	5-1	2-2
8-10	6-0	41.4	10	5	30	5-8	2-2
9-2	6-9	48.2	10	6	32	5-8	2-8
9-7	6-4	46.7	11	5	32	6-3	2-2
9-11	7-0	54.0	11	6	34	6-3	2-8
10-4	6-7	52.2	12	5	34	6-10	2-2
10-8	7-3	60.1	12	6	36	6-10	2-8
11-0	8-0	68.2	12	7	38	6-10	3-2
11-1	6-10	58.1	13	5	36	7-4	2-2
11-4	7-6	66.4	13	6	38	7-4	2-8
11-8	8-3	75.1	13	7	38	7-4	3-2
12-0	8-11	84.1	13	8	42	7-4	3-7
11-9	7-1	64.2	14	5	38	7-11	2-2
12-1	7-10	73.0	14	6	40	7-11	2-8
12-5	8-6	82.2	14	7	42	7-11	3-2
12-9	9-2	91.7	14	8	44	7-11	3-7
12-6	7-4	70.5	15	5	40	8-6	2-2
12-10	8-1	79.9	15	6	42	8-6	2-8
13-2	8-9	89.6	15	7	44	8-6	3-2
13-6	9-6	99.6	15	8	46	8-6	3-7
13-7	8-4	87.1	16	6	44	9-1	2-8
13-11	9-0	97.3	16	7	46	9-1	3-2
14-3	9-9	107.8	16	8	48	9-1	3-7
14-7	10-5	118.7	16	9	50	9-1	4-1
14-11	11-2	129.9	16	10	52	9-1	4-6

Note: Dimensions are to inside crest and are subject to manufacturing tolerances.
All dimensions, to the nearest whole number, are measured from inside crests.

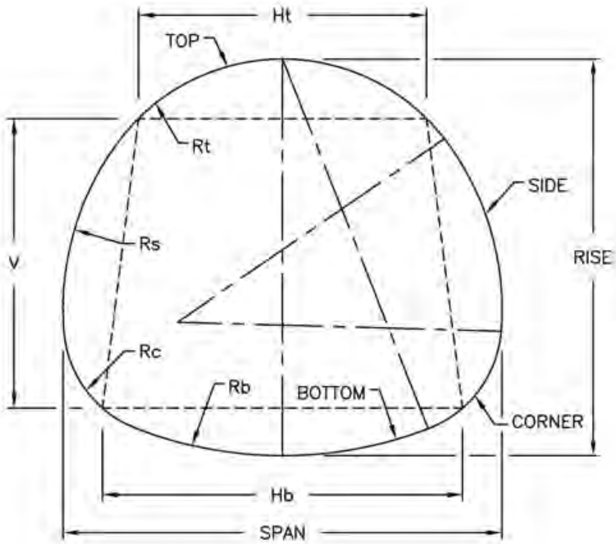


Table 2.31

Structural plate underpass — size and layout details
6 x 2 in. corrugation — bolted seams

Span x Rise (ft-in.)		Periphery		Clearance Box (ft- in.)			Layout Dimensions (in.)				
		Waterway Area (ft²)	N	HB	HT	V	No. of Plates per Ring	Top Radius R_t	Side Radius R_s	Corner Radius R_c	Bottom Radius R_b
5-8	5-9	27	24				6	27	53	18	Flat
5-8	6-6	32	26				6	29	75	18	Flat
5-9	7-4	36	28				6	28	95	18	Flat
5-10	7-8	38	29				7	30	112	18	Flat
5-10	8-2	41	30				6	28	116	18	Flat
8-6	8-6	58	35	4-6	4-6	7-6	7	44	96	31	144
8-8	8-8	62	36	5-0	5-0	7-6	7	47	96	31	191
8-11	8-11	65	37	6-0	6-0	7-2	7	49	100	31	243
9-8	9-4	73	39	6-0	6-0	7-9	7	53	84	38	191
10-10	9-6	81	41	7-0	7-0	7-5	7	59	96	38	123
11-5	10-3	93	44	8-0	8-0	7-6	7	66	100	38	174
12-2	11 -0	107	47	10-0	8-0	8-0	8	68	93	38	136
12-11	11 -2	116	49	10-0	8-0	8-6	9	74	92	38	148
13-2	11-10	126	51	10-0	8-0	9-6	11	73	102	38	161
13-10	12-2	136	53	10-0	8-0	10-0	11	77	106	38	168
14-1	12-10	147	55	12-0	10-0	9-0	11	77	115	38	183
14-6	13-5	158	57	12-0	10-0	9-6	11	78	131	38	174
14-10	14-0	169	59	12-0	10-0	10-6	11	79	136	38	193
15-6	14-4	180	61	12-0	10-0	11-0	12	83	139	38	201
15-9	15-1	192	63	12-0	10-0	12-0	12	82	151	38	212
16-4	15-5	204	65	12-0	10-0	12-6	12	86	156	38	217
16-5	16-0	217	67	12-0	10-0	13-0	12	88	159	38	271
16-9	16-3	224	68	12-0	10-0	13-6	12	89	168	38	246
17-3	17-0	239	70	12-0	10-0	14-0	12	90	174	47	214
18-4	16-11	252	72	16-0	12-0	12-0	12	99	157	47	248
19-1	17-2	266	74	16-0	12-0	13-0	13	105	156	47	262
19-6	17-7	280	76	16-0	12-0	13-6	13	107	158	47	295
20-4	17-9	295	78	16-0	12-0	14-0	13	114	155	47	316

Notes: Dimensions are to inside crests and are subject to manufacturing tolerances.
N = 3 Pi = 9.6 in.

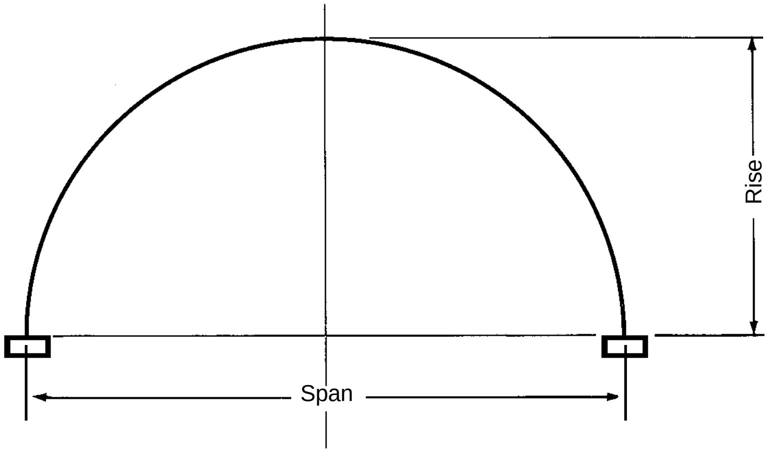


Table 2.32

Structural plate arch — representative sizes
6 x 2 in. corrugation — bolted seams

Dimensions (1) Span (ft)	Rise (ft-in.)	Waterway Area (ft^2)	Rise over Span	Radius (in.)	Nominal Arc Length N (2)
5.0	1-9 1/2	6.5	0.36		8
	2-2 1/2	8.5	0.44		9
	2-7 1/2	10.5	0.49		10
	3-0	12.4	0.60	30	11
6.0	1-9 1/2	7.5	0.30	41	9
	2-3 1/2	10.0	0.38	37 1/2	10
	3-2	15.0	0.53	36	12
	3-6	17.4	0.59	36	13
7.0	2-4	12.0	0.34	45	11
	2-10	15.0	0.40	43	12
	3-8	20.0	0.52	42	14
	4-1	23.1	0.58	42	15
	4-5	25.7	0.63	42	16
8.0	2-11	17.0	0.37	51	13
	3-4	20.0	0.42	48 1/2	14
	4-2	26.0	0.52	48	16
	4-7	29.7	0.57	48	17
	4-11	32.7	0.62	48	18
9.0	2-11	18.5	0.32	59	14
	3-10 1/2	26.5	0.43	54	16
	4-8 1/2	33.0	0.52	54	18
	5-1	37.1	0.57	54	19
	5-6	40.5	0.61	54	20
10.0	3-5 1/2	25.0	0.35	64	16
	4-5	34.0	0.44	60 1/2	18
	5-3	41.0	0.52	60	20
	5-7	45.3	0.56	60	21
	6-0	49.1	0.60	60	22
	6-4	52.8	0.64	60	23

Notes: (1) Dimensions are to inside crests and are subject to manufacturing tolerances.
(2) N = 3 Pi = 9.6 in.
Manufacturers may offer additional sizes.

Table 2.32 *continued*

Structural plate arch — representative sizes
6 x 2 in. corrugation — bolted seams

Dimensions (1) Span (ft)	Rise (ft-in.)	Waterway Area (ft^2)	Rise over Span	Radius (in.)	Nominal Arc Length N (2)
11.0	3-6	27.5	0.32	73	17
	4-5 1/2	37.0	0.41	67 1/2	19
	5-9	50.0	0.52	66	22
	6-1	54.3	0.56	66	23
	6-6	58.5	0.59	66	24
	6-11	62.7	0.63	66	25
12.0	4-0 1/2	35.0	0.34	77 1/2	19
	5-0	45.0	0.42	73	21
	6-3	59.0	0.52	72	24
	6-8	64.1	0.55	72	25
	7-0	68.8	0.59	72	26
	7-5	73.3	0.62	72	27
13.0	4-1	38.0	0.32	86 1/2	20
	5-1	49.0	0.39	80 1/2	22
	6-9	70.0	0.52	78	26
	7-2	74.8	0.55	78	27
	7-6	79.8	0.58	78	28
	7-11	84.8	0.61	78	29
14.0	4-7 1/2	47.0	0.33	91	22
	5-7	58.0	0.40	86	24
	7-3	80.0	0.52	84	28
	7-8	86.2	0.55	84	29
	8-1	91.7	0.58	84	30
	8-10	102.3	0.63	84	32
15.0	4-7 1/2	50.0	0.31	101	23
	5-8	62.0	0.38	93	25
	6-7	75.0	0.44	91	27
	7--9	92.0	0.52	90	30
	8-2	98.5	0.55	90	31
	8-7	104.4	0.57	90	32
	9-4	115.8	0.62	90	34
16.0	5-2	60.0	0.32	105	25
	7-1	86.0	0.45	97	29
	8-3	105.0	0.52	96	32
	8-8	111.6	0.54	96	33
	9-6	124.0	0.59	96	35
	9-10	130.1	0.62	96	36
17.0	5-2 1/2	63.0	0.31	115	26
	7-2	92.0	0.42	103	30
	8-10	119.0	0.52	102	34
	9-2	125.5	0.54	102	35
	10-0	138.7	0.59	102	37
	10-9	151.5	0.63	102	39
18.0	5-9	75.0	0.32	119	28
	7-8	104.0	0.43	109	32
	8-11	126.0	0.50	108	35
	9-9	140.2	0.54	108	37
	10-6	154.3	0.58	108	39
	11-3	167.9	0.63	108	41

Notes: (1) Dimensions are to inside crests and are subject to manufacturing tolerances.
(2) N = 3 Pi = 9.6 in.
Manufacturers may offer additional sizes.

Table 2.32 *continued*

Structural plate arch — representative sizes
6 x 2 in. corrugation — bolted seams

Dimensions [1] Span (ft)	Rise (ft-in.)	Waterway Area (ft^2)	Rise over Span	Radius (in.)	Nominal Arc Length N [2]
19.0	6-4	87.0	0.33	123	30
	8-2	118.0	0.43	115	34
	9-5 1/2	140.0	0.50	114	37
	10-3	155.8	0.54	114	39
	11-0	170.6	0.58	114	41
	11-10	185.1	0.62	114	43
20.0	6-4	91.0	0.32	133	31
	8-3 1/2	124.0	0.42	122	35
	10-0	157.0	0.50	120	39
	10-9	172.1	0.54	120	41
	11-6	187.8	0.58	120	43
	12-8	210.5	0.64	120	46
21.0	6-11	104.0	0.33	137	33
	8-10	140.0	0.42	128	37
	10-6	172.0	0.50	126	41
	11-3	189.3	0.54	126	43
	12-5	213.8	0.59	126	46
	13-3	229.7	0.63	126	48
22.0	6-11	109.0	0.31	146	34
	8-11	146.0	0.40	135	38
	11-0	190.0	0.50	132	43
	11-9	207.2	0.54	132	45
	13-0	233.0	0.59	132	48
	13-9	249.7	0.62	132	50
23.0	8-0	134.0	0.35	147	37
	9-10	171.0	0.43	140	41
	11-6	208.0	0.50	138	45
	12-8	235.1	0.55	138	48
	13-6	253.0	0.59	138	50
	14-8	279.0	0.64	138	53
24.0	8-6	150.0	0.35	152	39
	10-4	188.0	0.43	146	43
	12-0	226.0	0.50	144	47
	13-2	255.1	0.55	144	50
	14-0	273.8	0.58	144	52
	15-2	301.1	0.63	144	55
25.0	8-6 1/2	155.0	0.34	160	40
	10-10 1/2	207.0	0.43	152	45
	12-6	247.0	0.50	150	49
	13-9	275.9	0.55	150	52
	14-6	295.5	0.58	150	54
	15-8	324.0	0.63	150	57
26.0	8-0 1/2	149.0	0.31		40
	9-7	183.0	0.37		43
	10-11	214.0	0.42		46
	13-0	266.0	0.50		51
	14-3	297.6	0.55	156	54
	15-5	327.9	0.59	156	57
	16-7	357.3	0.64	156	60

Notes: (1) Dimensions are to inside crests and are subject to manufacturing tolerances.
(2) N = 3 Pi = 9.6 in.
Manufacturers may offer additional sizes.

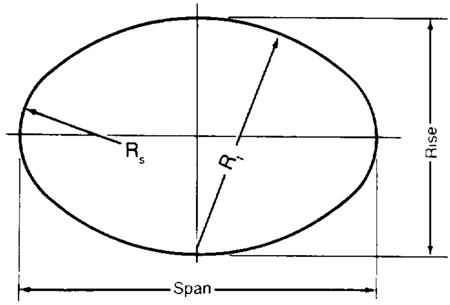


Table 2.33

Structural plate long span horizontal ellipse — sizes and layout details (1)
6 x 2 in. corrugation — bolted seams

Span (ft-in.)	Rise (ft-in.)	Area (ft²)	Periphery (2)			Inside Radius	
			Top or Bottom N	Side N	Total N	R_t Top Radius (in.)	R_s Side Radius (in.)
19-4	12-9	191	22	10	64	12-6	4-6
20-1	13-0	202	23	10	66	13-1	4-6
20-2	11-11	183	24	8	64	13-8	3-7
20-10	12-2	194	25	8	66	14-3	3-7
21- 0	15-2	248	23	13	72	13-1	5-11
21-11	13-1	221	26	9	70	14-10	4-1
22- 6	15-8	274	25	13	76	14-3	5-11
23- 0	14-1	249	27	10	74	15-5	4-6
23- 3	15-11	288	26	13	78	14-10	5-11
24- 4	16-11	320	27	14	82	15-5	6-4
24- 6	14-8	274	29	10	78	16-6	4-6
25- 2	14-11	287	30	10	80	17-1	4-6
25-5	16-9	330	29	13	84	16-6	5-11
26-1	18-2	369	29	15	88	16-6	6-10
26-3	15-10	320	31	11	84	17-8	4-11
27-0	16-2	334	32	11	86	18-3	4-11
27-2	19-1	405	30	16	92	17-1	7-3
27-11	19-5	421	31	16	94	17-8	7-3
28-1	17-1	369	33	12	90	18-10	5-5
28-10	17-5	384	34	12	92	19-5	5-5
29-5	19-11	455	33	16	98	18-10	7-3
30-1	20-2	472	34	16	100	19-5	7-3
30-3	17-11	415	36	12	96	20-7	5-5
31-2	21-2	512	35	17	104	20-0	7-9
31-4	18-11	454	37	13	100	21-1	5-11
32-1	19-2	471	38	13	102	21-8	5-11
32-3	22-2	555	36	18	108	20-7	8-2
33-0	22-5	574	37	18	110	21-1	8-2
33-2	20-1	512	39	14	106	22-3	6-4
34-1	23-4	619	38	19	114	21-8	8-8
34-7	20-8	548	41	14	110	23-5	6-4
34-11	21-4	574	41	1S	112	23-5	6-10
35-1	24-4	665	39	20	118	22-3	9-1
35-9	25-9	718	39	22	122	22-3	10-0
36-0	22-4	619	42	16	116	24-0	7-3
36-11	25-7	735	41	21	124	23-5	9-7
37-2	22-2	631	44	15	118	25-2	6-10
38-0	26-7	785	44	22	128	24-0	10-0
38-8	27-11	843	42	24	132	24-0	10-11
40-0	29-7	927	43	26	138	27-11	11-10

Notes: (1) Dimensions are to inside crests and are subject to manufacturing tolerances.
(2) N = 3 Pi = 9.6 in.
Manufacturers may offer additional sizes.

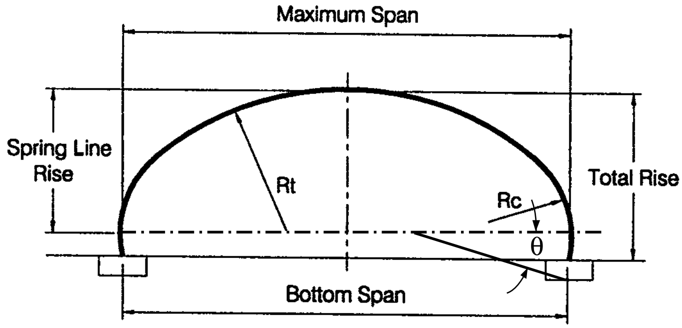


Table 2.34

Structural plate long span low profile arch — sizes and layout details (1)
6 x 2 in. corrugation — bolted seams

Max. Span (ft-in.)	Bottom Span (ft-in.)	Total Rise (ft-in.)	Area (ft^2)	Periphery (2) Top N	Side N	Total N	Inside Radius R_t Top Radius (in.)	R_s Side Radius (in.)
20-1	19-10	7-6	121	23	6	35	13-1	4-6
19-5	19-1	6-10	105	23	5	33	13-1	3-7
21-6	21-4	7-9	134	25	6	37	14-3	4-6
22-3	22-1	7-11	140	26	6	38	14-0	4-6
23-0	22-9	8-0	147	27	6	39	15-5	4-6
23-9	23-6	8-2	154	28	6	40	16-0	4-6
24-6	24-3	8-4	161	29	6	41	16-6	4-6
25-2	25-0	8-5	169	30	6	42	17-1	4-6
25-11	25-9	8-7	176	31	6	43	17-8	4-6
27-3	27-1	10-0	217	31	8	47	17-8	6-4
28-1	27-11	9-7	212	33	7	47	18-10	5-5
28-9	28-7	10-3	234	33	8	49	18-10	6-4
28-10	28-8	9-8	221	34	7	48	19-5	5-5
30-3	30-1	9-11	238	36	7	50	20-7	5-5
30-11	30-9	10-8	261	36	8	52	20-7	6-4
31-7	31-2	12-1	309	36	10	56	20-7	7-3
31-0	30-10	10-1	246	37	7	51	21-1	5-5
32-4	31-11	12-3	320	37	10	57	21-1	7-3
31-9	31-7	10-3	255	38	7	52	21-8	5-5
33-1	32-7	12-5	330	38	10	58	21-8	7-3
33-2	33-0	11-1	289	39	8	55	22-3	6-4
34-5	34-1	13-3	377	39	11	61	22-3	8-2
34-7	34-6	11-4	308	41	8	57	23-5	6-4
37-1	37-7	15-8	477	41	14	69	23-5	10-11
35-4	35-2	11-5	318	42	8	58	24-0	6-4
38-8	38-4	15-9	490	42	14	70	24-0	10-11

Notes: (1) Dimensions are to inside crests and are subject to manufacturing tolerances.
(2) N = 3 Pi = 9.6 in.
Manufacturers may offer additional sizes.

Low profile arch with concrete and bin-type retaining wall end treatment.

Long span high profile arch with concrete and bin-type retaining wall headwall.

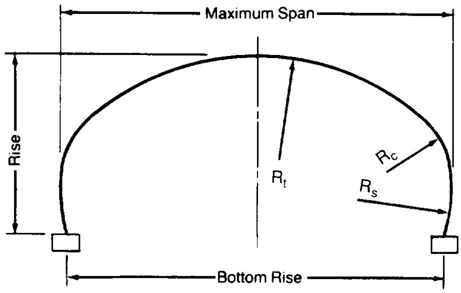


Table 2.35

Structural plate long span high profile arch — sizes and layout details (1)
6 x 2 in. corrugation — bolted seams

Max. Span (ft-in.)	Bottom Span (ft-in.)	Total Rise (ft-in.)	Area (ft^2)	Periphery (2) Top N	Upper Side N	Lower Side N	Total N	Inside Radius Top (ft-in.)	Upper Side (ft-in.)	Lower Side (ft-in.)
20-1	19- 6	9-1	152	23	5	3	39	13-1	4-6	13- 1
20-8	18-10	12-1	214	23	6	6	47	13- 1	5- 5	13- 1
21- 6	19-10	11-8	215	25	5	6	47	14- 3	4- 6	14- 3
22-10	19-10	14-7	285	25	7	8	55	14- 3	6- 4	14- 3
22- 3	20- 7	11-10	225	26	5	6	48	14-10	4- 6	14-10
22-11	20- 0	14-0	276	26	6	8	54	14-10	5- 5	14-10
23- 0	21- 5	12-0	235	27	5	6	49	15- 5	4- 6	15- 5
24- 4	21- 6	14-10	310	27	7	8	57	15- 5	6- 4	15- 5
23- 9	22- 2	12- 1	245	28	5	6	50	16- 0	4- 6	16- 0
24- 6	21-11	13- 9	289	29	5	8	55	16- 6	4- 6	16- 6
25- 9	23- 2	15- 2	335	29	7	8	59	16- 6	6- 4	16- 6
25- 2	23- 3	13- 2	283	30	5	7	54	17- 1	4- 6	17- 1
26- 6	24- 0	15- 3	348	30	7	8	60	17- 1	6- 4	17- 1
25-11	24- 1	13- 3	295	31	5	7	55	17- 8	4- 6	17- 8
27- 3	24-10	15- 5	360	31	7	8	61	17- 8	6- 4	17- 8
27- 5	25- 8	13- 7	317	33	5	7	57	18-10	4- 6	18-10
29- 5	27- 1	16- 5	412	33	8	8	65	18-10	7- 3	18-10
28- 2	25-11	14- 5	349	34	5	8	60	19- 5	4- 6	19- 5
30- 1	26- 9	18- 1	467	34	8	10	70	19- 5	7- 3	19- 5
30- 3	28- 2	15- 5	399	36	6	8	64	20- 7	5- 5	20- 7
31- 7	28- 4	18- 4	497	36	8	10	72	20- 7	7- 3	20- 7
31- 0	29- 0	15- 7	413	37	6	8	65	21- 1	5- 5	21- 1
31- 8	28- 6	17- 9	484	37	7	10	71	21- 1	6- 4	21- 1
32- 4	27-11	19-11	554	37	8	12	77	21- 1	7- 3	21- 1
31- 9	28- 8	17- 3	470	38	6	10	70	21- 8	5- 5	21- 8
33- 1	28- 9	20- 1	571	38	8	12	78	21- 8	7- 3	21- 8
32- 6	29- 6	17- 4	484	39	6	10	71	22- 3	5- 5	22- 3
33-10	29- 7	20- 3	588	39	8	12	79	22- 3	7- 3	22- 3
34- 0	31- 2	17- 8	514	41	6	10	73	23- 5	5- 5	23- 5
34- 7	30- 7	19-10	591	41	7	12	79	23- 5	6- 4	23- 5
35- 3	30- 7	21- 3	645	41	8	13	83	23- 5	7- 3	23- 5
37- 3	32- 6	23- 5	747	41	11	13	89	23- 5	10- 0	23- 5
34- 8	31-11	17-10	529	42	6	10	74	24- 0	5- 5	24- 0
35- 4	31- 5	20- 0	608	42	7	12	80	24- 0	6- 4	24- 0
36- 0	31- 5	21- 5	663	42	8	13	84	24-0	7- 3	24- 0
38- 0	33- 5	23- 6	767	42	11	13	90	24- 0	10- 0	24- 0

Notes: (1) Dimensions are to inside crests and are subject to manufacturing tolerances.
(2) N = 3 Pi = 9.6 in.
Manufacturers may offer additional sizes.

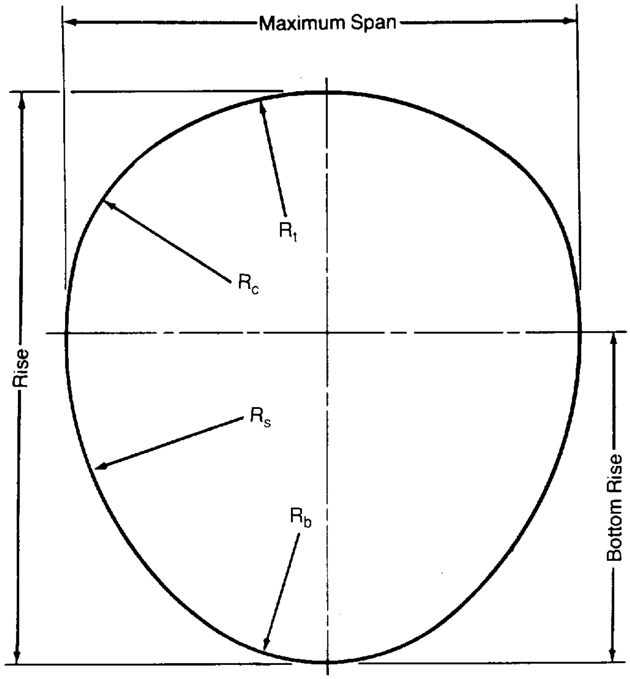


Table 2.36

Structural plate long span pear shape — sizes and layout details [1]
6 x 2 in. corrugation — bolted seams

Max. Span (ft-in.)	Rise (ft-in.)	Bottom Rise (ft-in.)	Area (ft²)	Periphery (2) Top N	Corner N	Side N	Bottom N	Total N	Inside Radius Bottom (ft-in.)	Side (ft-in.)	Corner (ft-in.)	Top (ft-in.)
23-8	25-8	14-11	481	25	5	24	15	98	8-11	16-7	6-3	14-8
24-0	25-10	15-1	496	22	7	22	20	100	9-11	17-4	7-0	16-2
25-6	25-11	15-10	521	27	7	20	21	102	10-7	18-1	6-11	15-10
24-10	27-8	16-9	544	27	5	25	18	105	9-3	19-8	5-9	15-11
27-5	27-0	18-1	578	30	6	26	16	110	9-7	20-4	4-7	19-11
26-8	28-3	18-0	593	28	5	30	12	110	8-0	20-1	4-9	20-11
28-1	27-10	16-10	624	27	8	22	25	112	12-2	19-0	7-3	20-5
28-7	30-7	19-7	689	32	7	24	24	118	11-2	24-0	7-0	18-2
30-0	29-8	20-0	699	32	8	23	25	119	11-11	24-0	6-7	21-10
30-0	31-2	19-11	736	34	7	24	26	122	12-1	24-0	7-0	19-3

Notes: (1) Dimensions are to inside crests and are subject to manufacturing tolerances.
(2) N = 3 Pi = 9.6 in.
Manufacturers may offer additional sizes.

Bolts and Nuts

Galvanized 3/4 inch or 7/8 inch diameter bolts of special heat-treated steel meeting ASTM Specification A 449 or ASTM Specification F568 Class 8.8, are used to assemble structural plate sections. Galvanized nuts meet the requirements of ASTM A 563 Grade 12. The galvanizing on bolts and nuts must meet ASTM Specification A 153, Class 5 or ASTM B 695 Class 50 Type II. See Figure 2.6 for dimensions of bolts and nuts. Lengths include: 1 1/4, 1 1/2, 1 3/4, 2, 3 and 4 inches. The containers and bolts may be color coded for ease in identification. These are designed for fitting either the crest or valley of the corrugations, and to give maximum bearing area and tight seams without the use of washers. Power wrenches are generally used for bolt tightening, but simple hand wrenches are satisfactory for small structures.

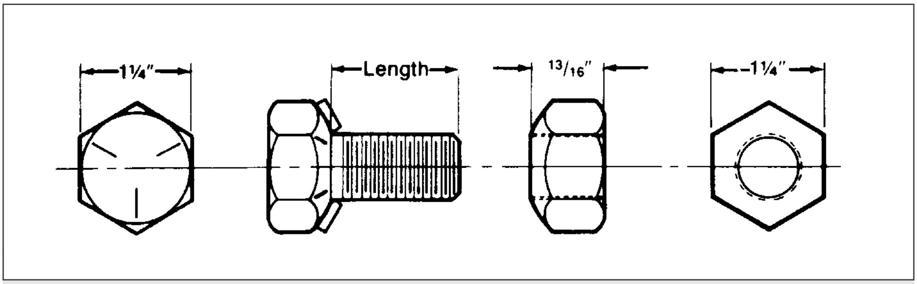


Figure 2.6 Dimensions of bolts and nuts for structural plate. Lengths include: 1 1/4 in., 1 1/2 in., 1 3/4 in., 2 in., 3 in. and 4 in.

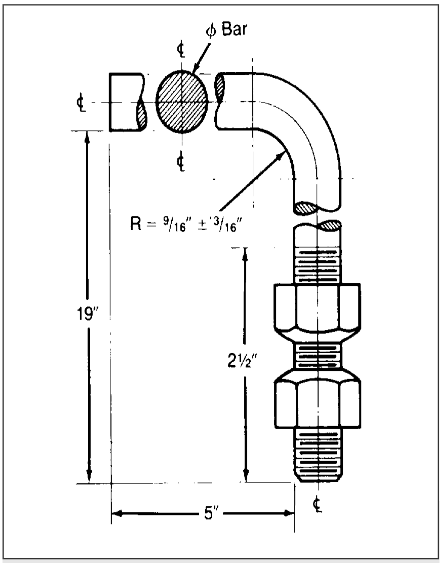


Figure 2.7 Hook bolts and nuts for embedment in headwalls.

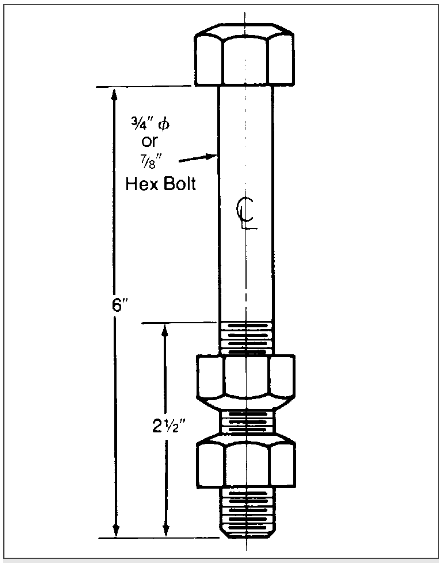


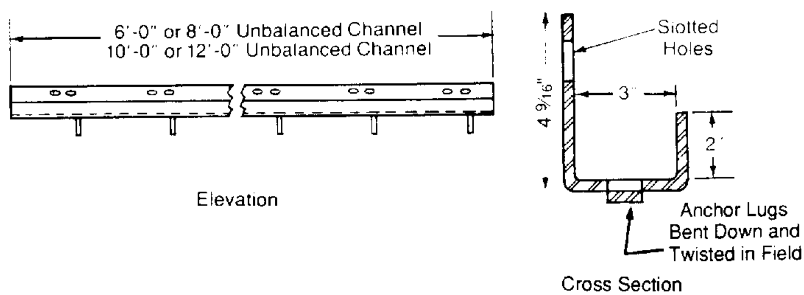
Figure 2.8 Straight anchor bolt.

Anchor bolts are available for anchoring the sides of structural plate arches into footings, and the ends of structural plate pipe into concrete end treatments Material for these special 3/4 inch or 7/8 inch bolts must conform to ASTM Specification A 307, and nuts to ASTM A 563 Grade C. Galvanizing of anchor bolts and nuts must conform to ASTM A 153.

■ **Figure 2.9** High-strength steel bolts are used for the circumferential and longitudinal seams of structural plate pipe. Four, six, or eight bolts per foot of longitudinal seam provide the strength required for the loading conditions.

Arch Channels

For arch seats, galvanized unbalanced channels are available for anchoring the arch to concrete footings. The unbalanced channel is anchored to the footing either by anchor bolts or by integral lugs that are bent and twisted as shown in Figure 2.10.



■ **Figure 2.10** General dimensions of unbalanced channels for 6 x 2 in. structural plate arches.

DEEP CORRUGATED STRUCTURAL PLATE

Deep corrugated structural plate pipe is also a bolted structure. It has either a 15 x 5 1/2 inch corrugation (DCSP Type I) or a 16 x 6 inch corrugation (DCSP Type II). As with standard (6 x 2 inches) structural plate, the corrugations are at right angles to the length of the structure. The length of a plate is measured in a direction parallel to the length of the structure. The width of a plate is, therefore, measured in a direction that is perpendicular to the length of the structure, around the periphery or circumference of the structure.

DEEP CORRUGATED STRUCTURAL PLATE TYPE I

Product Description

Deep corrugated structural plate pipe Type I has a 15 x 5 1/2 in. corrugation, which is shown in Figure 2.1. Standard plates are fabricated in one length and 12 different widths, as shown in Table 2.37 and Figure 2.11. The coverage length (excluding the side laps) is 30 inches The plate width designation, S, is used to describe the various plate widths available. S is the distance between circumferential bolt holes, or one circumferential bolt hole space (circumferential refers to the direction around the periphery of the structure, at right angles to the length of the structure). For instance, a 5 S plate has a net width of 5 circumferential bolt hole spaces. The bolt hole space, S, is 16 inches.

Plates are furnished curved to various radii and are clearly identified and located on the assembly drawings provided by the fabricator for field erection. The plates are available in 0.140 to 0.315 inch thicknesses. Weights of individual plate sections are shown in Table 2.38.

Section Properties

Section properties, used for design, are provided in Table 2.14. Properties of the arc-and-tangent corrugation are derived mathematically using the design thickness. The properties in the table include area, moment of inertia, section modulus and radius of gyration.

Sizes and Shapes

The plates are assembled into various shapes as indicated in Tables 2.39 through 2.41. The shapes include round, single-radius arch, multi-radius arch, and box culvert. Special shapes, and other standard shape sizes not shown in the tables, are also available. See Figures 2.12 - 2.14 for additional details. Detailed assembly instructions accompany each structure.

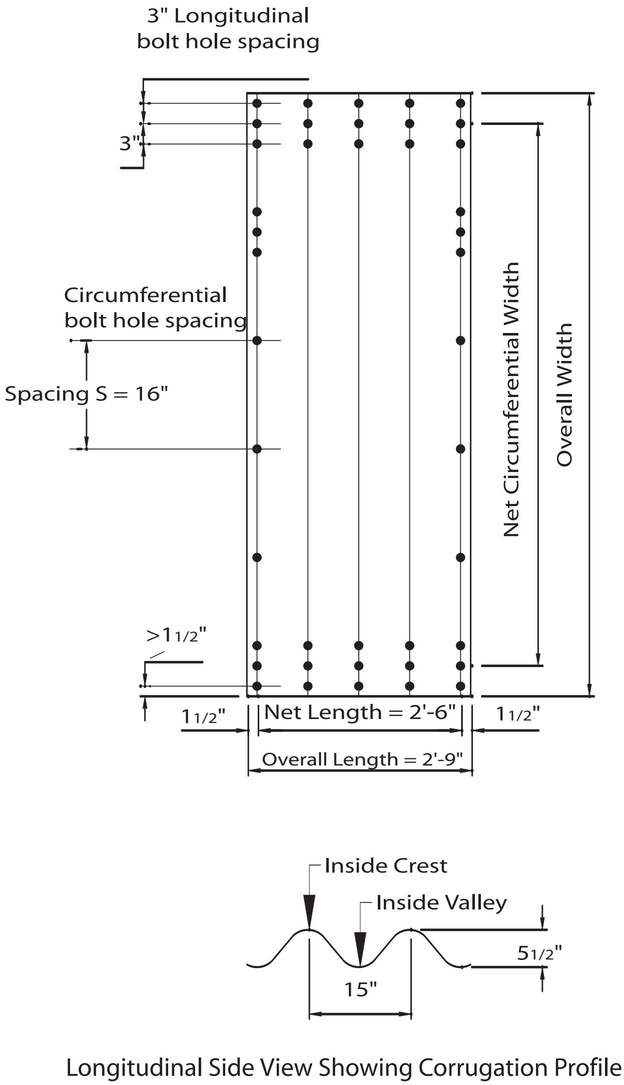


Figure 2.11 Deep corrugated structural plate configuration.

Table 2.37

15 x 5 1/2 in. deep corrugated structural plate sections
Details of uncurved plates

Nominal Plate Width Designation, *S	Net Width, in.	Overall Width, in.	No. of Circumferential Bolt Holes
1S	16	25	2
2S	32	41	3
3S	48	57	4
4S	64	73	5
5S	80	89	6
6S	96	105	7
7S	112	121	8
8S	128	137	9
9S	144	153	10
10S	160	169	11
11S	176	185	12
12S	192	201	13

*S = 16 inches

Table 2.38

Weight of 15 x 5 1/2 in. deep corrugated structural plate sections

Plate Width Designation, *S	Net Length, ft	Approx. Wt. Of Individual Galvanized Plates Without Bolts, in Pounds Specified Thickness, in.						Number of Assembly Bolts/Plate
		0.140	0.170	0.188	0.218	0.249	0.280	
1S	2.5	45	54	60	71	81	91	14
2S	2.5	73	88	98	116	132	149	15
3S	2.5	102	122	136	161	184	207	16
4S	2.5	130	156	174	206	235	265	17
5S	2.5	159	190	212	251	286	323	18
6S	2.5	187	224	250	296	338	381	19
7S	2.5	216	258	287	341	389	439	20
8S	2.5	244	292	325	385	440	497	21
9S	2.5	272	326	363	430	492	555	22
10S	2.5	301	360	401	475	543	613	23
11S	2.5	329	394	439	520	594	671	24
12S	2.5	358	428	477	565	646	729	25

Notes:
1 Bolts are color coded for the different lengths.
2 Weight of bolts in pounds per hundred pieces:
2 in. = 59.5
3 in. = 72.5
4 in. = 85.5
3 To compute the approximate weight of structures per foot of length: (1) multiply the number of plates in the periphery by the plate weights from the table; (2) add weight of the bolts; (3) Divide by plate length.

*S = 16 in.

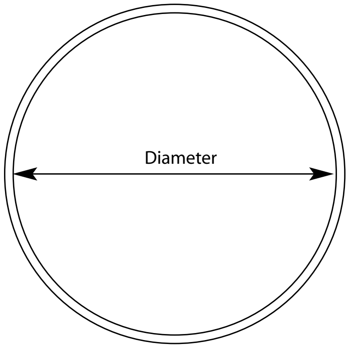


Table 2.39

Structural plate long span round [1]
15 x 5 1/2 in. corrugation — bolted seams

Inside Diameter ft - in.	Periphery S*	End Area ft^2
27 - 6	66	596
28 - 4	68	634
29 - 3	70	672
30 - 1	72	712
30 - 11	74	752
31 - 9	76	794
32 - 7	78	837
33 - 5	80	881
34 - 4	82	926
35 - 2	84	973
36 - 0	86	1020
36 - 10	88	1069
37 - 8	90	1119
39 - 5	94	1221
41 - 1	98	1329
42 - 10	102	1441
44 - 6	106	1557
46 - 2	110	1678
47 - 11	114	1804
49 - 7	118	1934
51 - 3	122	2069

Note: 1. All dimensions are to the inside crest and subject to manufacturing tolerances.
2. Other sizes are available.
3. All structures should be reviewed based on live load and geotechnical conditions

*S = 16 in.

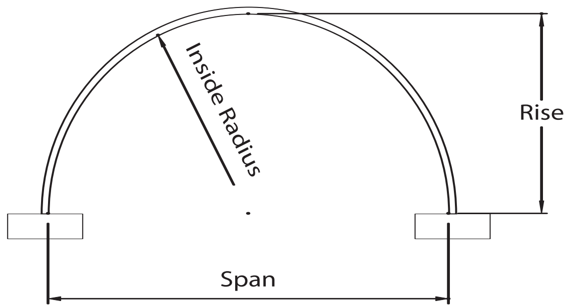


Table 2.40

Deep Corrugated Arches — Sizes and layout details (1)
15 x 5-1/2 in corrugation profile — bolted seams

Span (ft-in.)	Total Rise (ft-in.)	End Area (ft^2)	Inside Radius (ft-in.)	Total S*
22' 11"	11' 5"	207	11' 5"	27
23' 9"	11' 11"	222	11' 11"	28
24' 8"	12' 4"	238	12' 4"	29
25' 1"	12' 6"	255	12' 9"	30
26' 4"	13' 2"	272	13' 2"	31
27' 2"	13' 7"	291	13' 7"	32
28' 1"	14' 0"	309	14' 0"	33
28' 10"	14' 5"	327	14' 5"	34
29' 9"	14' 10"	347	14' 10"	35
30' 7"	15' 3"	367	15' 3"	36
31' 5"	15' 9"	387	15' 9"	37
32' 3"	16' 2"	409	16' 2"	38
33' 2"	16' 6"	431	16" 7"	39
34' 0"	17' 0"	453	17' 0"	40
35' 8"	17' 10"	499	17' 10"	42
37' 4"	18' 8"	548	18' 6"	44
39' 1"	19' 6"	600	19' 6"	46
40' 9"	20' 4"	652	20' 4"	48
42' 6"	21' 3"	708	21' 3"	50
44' 2"	22' 1"	765	22' 1"	52
45' 10"	22' 11"	826	22' 11"	54
49' 3"	24' 11"	953	24' 7"	58
50' 11"	25' 6"	1019	25' 6"	60
52' 8"	26' 4"	1088	26' 4"	62
54' 8"	27' 2"	1159	27' 2"	64
56' 6"	28' 4"	1234	28' 3"	66
57' 8"	28' 10"	1309	28' 10"	68
59' 5"	29' 9"	1387	29' 9"	70
62' 10"	31' 5"	1677	31' 5"	74
66' 3"	33' 1"	1722	33' 2"	78
67' 11"	34' 0"	1812	34' 0"	80
69' 7"	34' 10"	1903	34' 9"	82
73' 0"	36' 6"	2094	36' 6"	86
74' 8"	37' 4"	2191	37' 4"	88
78' 9"	39' 6"	2448	39' 4"	93
82' 0"	41' 0"	2641	41' 0"	96

Note: 1. All dimensions are to the inside crest and subject to manufacturing tolerances.
2. Other sizes are available.
3. All structures should be reviewed based on live load and geotechnical conditions.

*S = 16 in.

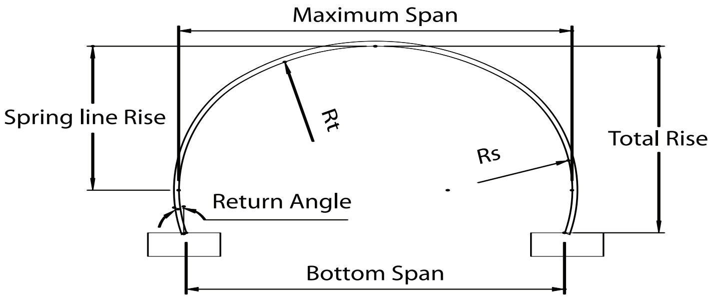


Table 2.41

Structural plate multi-radius arches — size and layout details
15 x 5-1/2 in. corrugations — bolted seams

Max Span (ft-in.)	Bottom Span (ft-in.)	Total Rise (ft-in.)	End Area (ft^2)	Inside Radius Side (in.)	Inside Radius Crown (in.)	Return Angle degrees	Total S*
26' 3"	26' 3"	11' 9"	253.3	135	391	2.3	30
29' 6"	29' 6"	12' 4"	303.0	135	391	0	33
29' 3"	28' 7"	16' 11"	437.8	135	391	9.6	40
31' 2"	31' 0"	13' 0"	339.5	135	391	6.5	35
32' 10"	32' 8"	13' 0"	356.7	135	391	5.7	36
32' 10"	31' 11"	14' 11"	419.0	135	391	15.6	39
32' 10"	31' 9"	17' 7"	506.5	135	391	10.1	43
34' 5"	34' 4"	13' 9"	374.0	135	391	4.8	37
36' 1"	35' 11"	13' 9"	414.7	135	391	10.1	39
36' 1"	35' 1"	15' 8"	482.9	135	391	0	42
36' 1"	35' 3"	19' 3"	605.7	147	391	10.1	47
37' 9"	37' 7"	13' 10"	433.3	135	391	6.1	40
39' 4"	39' 3"	14' 0"	452.3	135	391	5	41
39' 4"	38' 8"	18' 6"	614.6	174	391	12.8	47
39' 4"	38' 0"	20' 0"	685.4	147	391	11.3	50
41' 0"	40' 10"	14' 9"	497.9	135	391	7.2	43
42' 8"	42' 6"	14' 11"	518.6	135	391	5.9	44
42' 8"	41' 10"	19' 4"	693.1	174	391	13.8	50
42' 8"	41' 6"	21' 2"	775.7	163	391	10.6	53
44' 3"	44' 2"	15' 2"	539.5	135	391	4.5	45
45' 11"	45' 10"	16' 0"	590.7	135	391	0	47
45' 11"	45' 6"	21' 6"	817.1	214	391	0	54
45' 11"	44' 9"	23' 0"	899.5	178	391	10.9	57
47' 7"	47' 4"	16' 11"	644.4	135	391	8	49
49' 3"	49' 1"	17' 2"	669.0	135	391	6.2	50
49' 3"	48' 6"	23' 0"	939.9	214	391	11.6	58
49' 3"	48' 1"	24' 1"	999.9	186	391	10.6	60
50' 10"	50' 8"	18' 1"	727.1	135	391	7.5	52
52' 6"	52' 4"	16' 2"	693.9	135	548	7.5	52
52' 6"	52' 1"	21' 10"	962.4	214	548	8.9	59
52' 6"	51' 0"	26' 2"	1195.9	194	548	10.4	66
54' 2"	53' 10"	16' 11"	751.1	135	548	9.5	54
55' 9"	55' 7"	17' 2"	775.2	135	548	8.2	55
55' 9"	55' 6"	22' 1"	1022.1	214	548	7.6	61
55' 9"	54' 0"	27' 10"	1345.2	202	548	11.1	70

Note: 1. All dimensions are to the inside crest and subject to manufacturing tolerances.
2. Other sizes are available.
3. All structures should be reviewed based on live load and geotechnical conditions.

*S = 16 in.

Table 2.41 *continued*

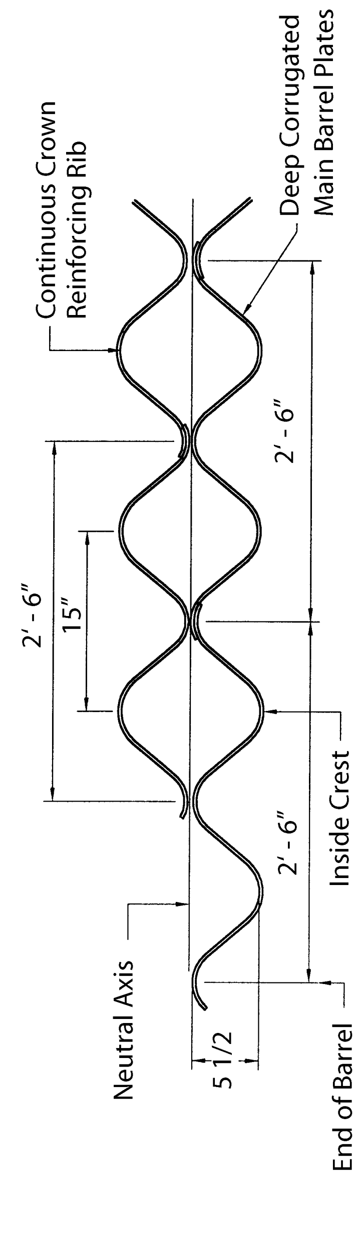
Structural plate multi-radius arches — size and layout details
15 x 5-1/2 in. corrugations — bolted seams

Max Span (ft-in.)	Bottom Span (ft-in.)	Total Rise (ft-in.)	End Area (ft²)	Inside Radius Side (in.)	Inside Radius Crown (in.)	Return Angle degrees	Total S*
57' 5"	57' 3"	17' 4"	799.6	135	548	6.8	56
59' 1"	58' 10"	18' 2"	862.6	135	548	8.7	58
59' 1"	58' 8"	23' 0"	1121.2	214	548	8.3	64
60' 8"	60' 6"	18' 5"	889.0	135	548	7.1	59
62' 4"	62' 1"	19' 4"	956.0	135	548	8.8	61
62' 4"	62' 1"	23' 3"	1185.1	214	548	6.7	66
64' 0"	63' 10"	19' 7"	984.4	135	548	7.1	62
65' 7"	65' 4"	20' 6"	1055.9	135	548	8.6	64
65' 7"	65' 4"	24' 4"	1293.5	214	548	7	69
67' 3"	67' 1"	20' 10"	1086.5	135	548	6.6	65
68' 11"	68' 6"	27' 9"	1553.5	253	548	7.4	75
70' 6"	70' 4"	22' 9"	1240.4	135	548	8.8	69
72' 2"	71' 11"	19' 5"	1121.5	135	745	8.6	68
72' 2"	71' 10"	26' 9"	1607.2	253	745	7.7	77
73' 10"	73' 5"	20' 3"	120.2	135	745	10.5	70
75' 5"	75' 1"	22' 10"	1394.2	174	745	8.9	74
75' 5"	74' 11"	29' 3"	1837.1	273	745	8.9	82
77' 0"	76' 10"	23' 0"	1426.6	174	745	7.8	75
78' 9"	78' 4"	23' 10"	1510.6	174	745	9.3	77
78' 9"	78' 4"	29' 6"	1918.0	273	745	7.8	84
80' 5"	80' 0"	24' 0"	1545.0	174	745	8.1	78
82' 0"	81' 10"	24' 4"	1580.0	174	745	6.9	79
82' 0"	81' 6"	30' 6"	2053.2	273	745	8.2	87
83' 8"	83' 4"	25' 2"	1669.4	174	745	8.2	81

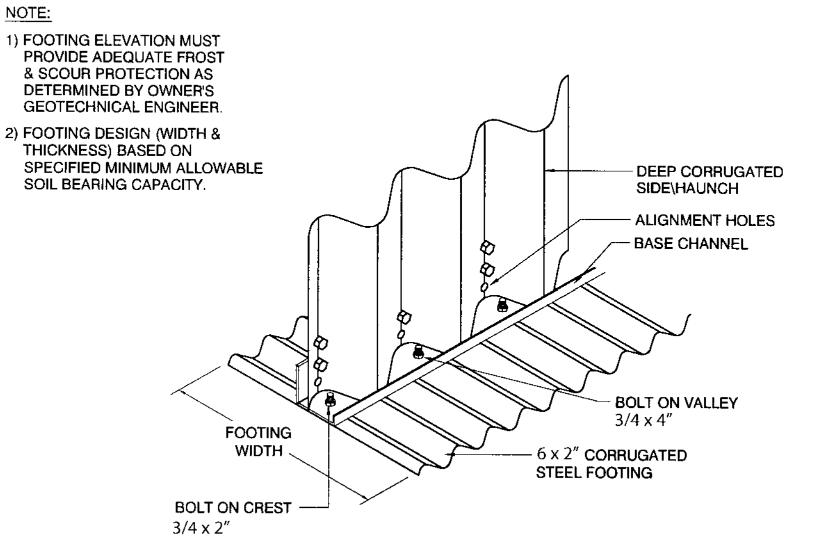
Note: 1. All dimensions are to the inside crest and subject to manufacturing tolerances.
2. Other sizes are available.
3. All structures should be reviewed based on live load and geotechnical conditions.

*S = 16 in.

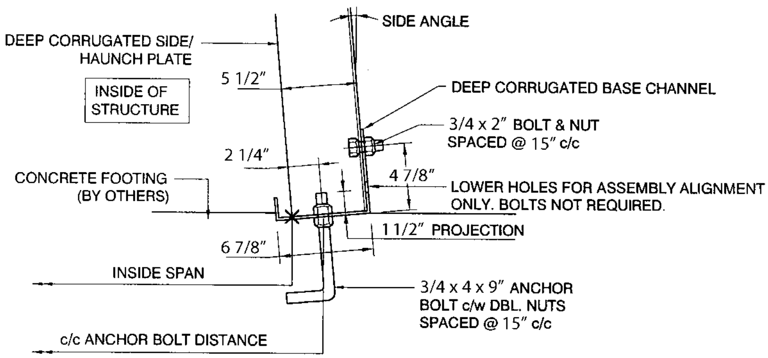
35 foot span by 15 foot rise deep corrugated structural plate arch with beveled ends for a Fish Passage Project in the Willamette National Forest in Oregon.

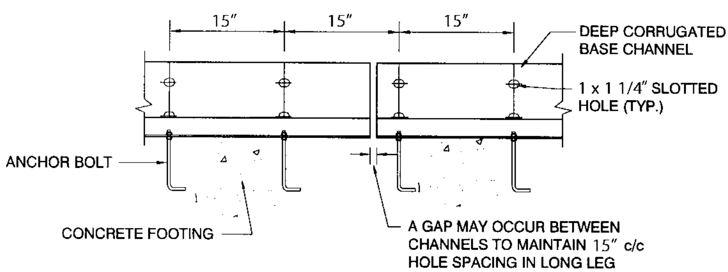


■ **Figure 2.12** Reinforcing rib for deep corrugated structural plate 15 x 5 1/2 in. corrugation profile.



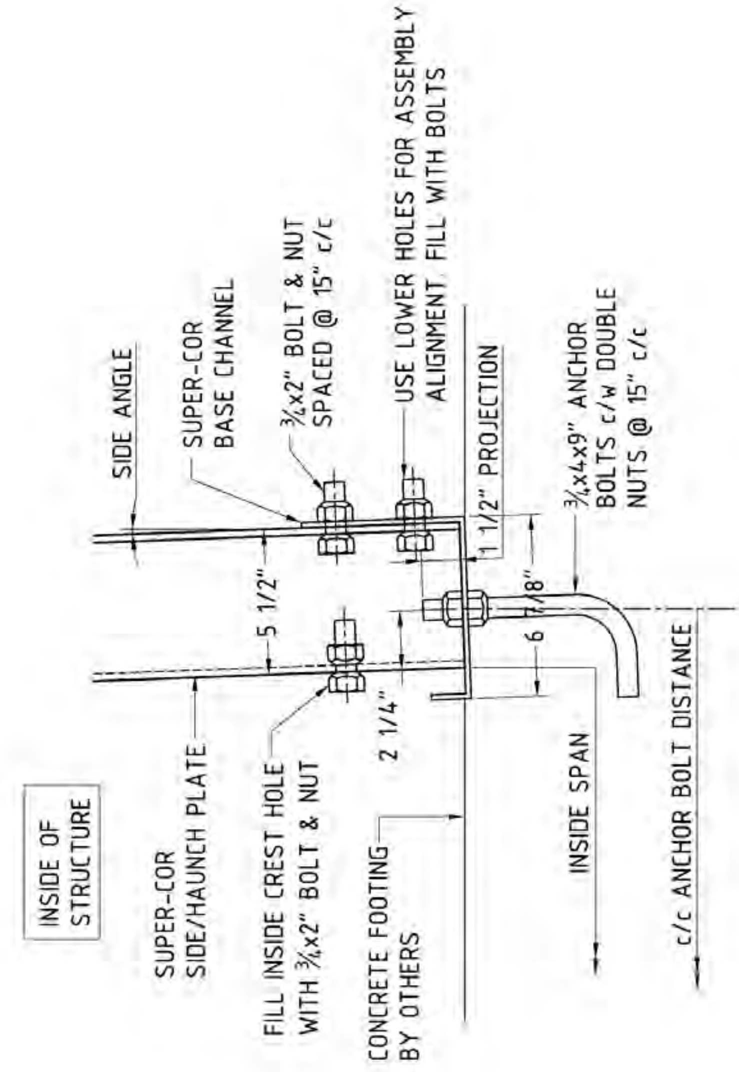
Detail – Typical Steel Footer





Details – Base Channel

■ **Figure 2.13** Additional details for deep corrugated structural plate.



■ **Figure 2.14** General dimensions of unbalanced channel for 15 x 5 1/2 in. structural plate arches.

DEEP CORRUGATED STRUCTURAL PLATE TYPE II

Product Description

Deep corrugated structural plate Type II has a 16 x 6 inch corrugation, which is shown in Figure 2.1.

Standard plates are fabricated in one length and several widths, as shown in Table 2.42 and Figure 2.15. The coverage length (excluding the side laps) is 47 1/4 inches The plate width designation, H, is used to describe the various plate widths available. H is the distance between circumferential bolt holes, or one circumferential bolt hole space (circumferential refers to the direction around the periphery of the structure, at right angles to the length of the structure). For instance, a 9 H plate has a net width of 9 circumferential bolt hole spaces (see Figure 2.15). The bolt hole space, H, is 16 3/4 inches.

Plates are furnished curved to various radii and are identified with a permanent mark. This marking is provided to simplify field erection and to make identification of the structure details, in the future, as easy as possible. The fabricator provides field assembly drawings to guide the installer. The plates are available in thickness ranging from 0.169 to 0.315 inches Weights of individual plate sections are shown in Table 2.42.

Table 2.42

Weight of 16 x 6 in. deep corrugated structural plate sections

Plate Width Designation, 'H' ('H'=16.75")	Net Length ft	Approx. Wt. Of Individual Galvanized Plates Without Bolts, in Pounds — Specified Thickness, in.					Number of Assembly Bolts/Plate
		0.169	0.197	0.236	0.276	0.315	
4	3.94	239	282	341	398	451	23
5	3.94	291	342	415	484	548	24
6	3.94	342	402	488	569	645	25
7	3.94	394	463	561	655	742	26
8	3.94	445	523	634	740	839	27
9	3.94	496	584	708	825	936	28
10	3.94	548	644	781	911	1033	29

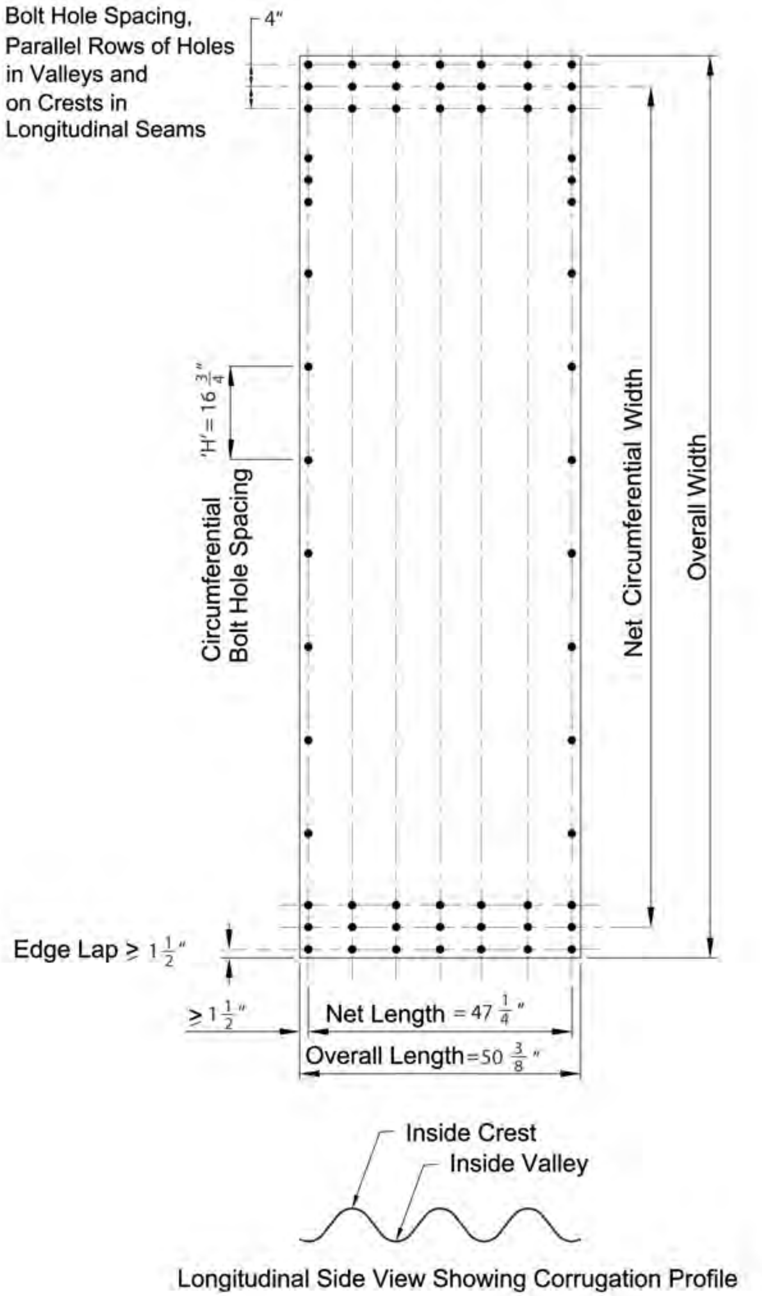
Notes: 1. Bolt lengths used for all structures = 2"
2. Weight of bolts and nuts in pounds per hundred = 59.5 lbs.
3. To compute the approximate weight of structure per foot of length: (1) Multiply the number of plates in the periphery by the plate weights in the table; (2) add weight of bolts; (3) divide by the net plate length.

Table 2.43

16 x 6 in. deep corrugated structural plate sections
Details of uncurved plates

Nominal Plate Width Designation, 'H'*	Net Width, in.	Overall Width, in.	No. of Circumferential Bolt Holes
3H	50.2	61.22	4
4H	66.93	77.95	5
5H	83.67	94.69	6
6H	100.40	111.42	7
7H	117.13	128.15	8
8H	133.86	144.88	9
9H	150.59	161.61	10
10H	167.32	178.35	11

Note: DCSP Type ll is a metric profile. Values shown are hard converted on the basis 1.0 in. = 25.4 mm
*'H' = 16.75 in.



■ **Figure 2.15** 16 x 6 in. deep corrugated structural plate

Section Properties

Section properties, used for design, are provided in Table 2.15. Properties of the arc-and-tangent corrugation are derived mathematically using the design thickness. The properties in the table include area, moment of inertia, section modulus and radius of gyration.

Sizes and Shapes

The plates are assembled into various shapes as indicated in Tables 2.44 through 2.46. The shapes include round, single-radius arch, two radius arches, and box culverts. Special shapes, and other standard shape sizes not shown in the tables, are also available. See Figures 2.16 - 2.18 for additional details. Detailed assembly instructions accompany each structure.

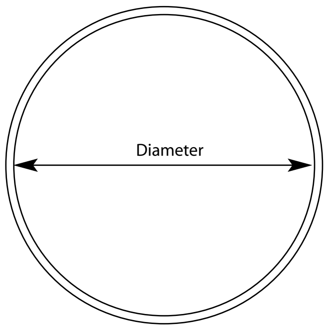


Table 2.44

Structural plate corrugated steel pipe
16 x 6 in. corrugations — bolted seams

Inside Diameter ft - in.	Total Periphery H*	End Area ft^2
19 - 11	46	312
20 - 10	48	340
21 - 8	50	370
22 - 7	52	401
23 - 6	54	433
24 - 4	56	466
25 - 3	58	501
26 - 2	60	537
27 - 0	62	574
27 - 11	64	612
28 - 10	66	652
29 - 8	68	692
30 - 7	70	734
31 - 6	72	778
32 - 4	74	822
33 - 3	76	868
34 - 1	78	915

*H = 16.75 in.
Notes: 1. All dimensions are to the inside crest and are subject to manufacturing tolerances.
2. Sizes are representative, other sizes may be available, contact your manufacturer.

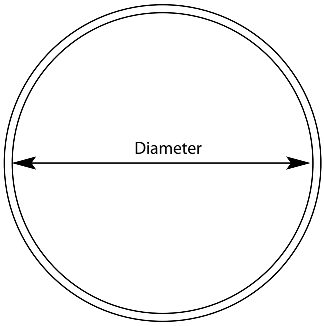


Table 2.44 *continued*

Structural plate corrugated steel pipe
16 x 6 in. corrugations — bolted seams

Inside Diameter ft - in.	Total Periphery H*	End Area ft^2
35 - 0	80	963
35 - 11	82	1012
36 - 10	84	1063
37 - 8	86	1115
38 - 7	88	1168
39 - 5	90	1223
40 - 4	92	1278
41 - 3	94	1335
42 - 1	96	1393
43 - 0	98	1452
43 - 11	100	1513
44 - 9	102	1575
45 - 8	104	1638
46 - 7	106	1702
47 - 5	108	1768
48 - 4	110	1834
49 - 3	112	1902
50 - 1	114	1972
51 - 0	116	2042
51 - 11	118	2114

*H = 16.75 in.
Notes: 1. All dimensions are to the inside crest and are subject to manufacturing tolerances.
2. Sizes are representative, other sizes may be available, contact your manufacturer.

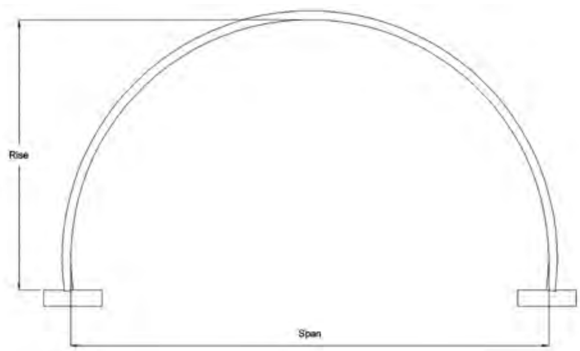


Table 2.45

Structural plate single-radius arches — size and layout details
16 x 6 in. corrugations — bolted seams

Span ft-in.	Rise ft-in.	End Area ft^2	Inside Radius in.	Total Periphery H*
26 – 3	12 - 10	262.2	157	29
27 – 1	13 - 3	280.5	162	30
27 – 11	13 - 9	299.4	167	31
28 – 8	14 - 2	319.0	172	32
29 – 6	14 - 8	339.2	177	33
30 – 4	15 - 1	359.9	182	34
31 – 2	15 - 7	381.4	187	35
31 – 12	16 - 1	403.3	192	36
32 – 10	16 - 6	425.9	197	37
33 – 8	16 - 12	449.2	202	38
34 – 5	17 - 5	473.1	207	39
35 – 3	17 - 11	497.5	212	40
36 – 1	18 - 4	522.6	217	41
36 – 11	18 - 10	548.3	221	42
37 – 9	18 - 7	548.6	226	42
38 – 7	19 - 1	575.0	231	43
39 – 4	19 - 6	602.0	236	44
40 – 2	19 - 12	629.6	241	45
41 – 0	20 - 5	657.8	246	46
41 – 10	20 - 11	686.6	251	47
42 – 8	21 - 4	716.0	256	48
43 – 6	21 - 10	746.0	261	49
44 – 3	22 - 4	776.7	266	50
45 – 1	22 - 9	808.0	271	51
45 – 11	23 - 3	839.9	276	52
46 – 9	23 - 8	872.4	281	53
47 – 7	24 - 2	905.5	285	54
48 – 5	24 - 7	939.2	290	55
49 – 3	24 - 5	942.9	295	55
50 – 0	24 - 10	974.0	300	56
51 – 0	25 - 4	1009.1	305	57
51 – 8	25 - 9	1044.7	310	58
52 – 6	26 - 3	1081.0	315	59

*H = 16.75 in.
Notes: 1. All dimensions are to the inside crest and are subject to manufacturing tolerances.
2. Sizes are representative, other sizes may be available, contact your manufacturer.

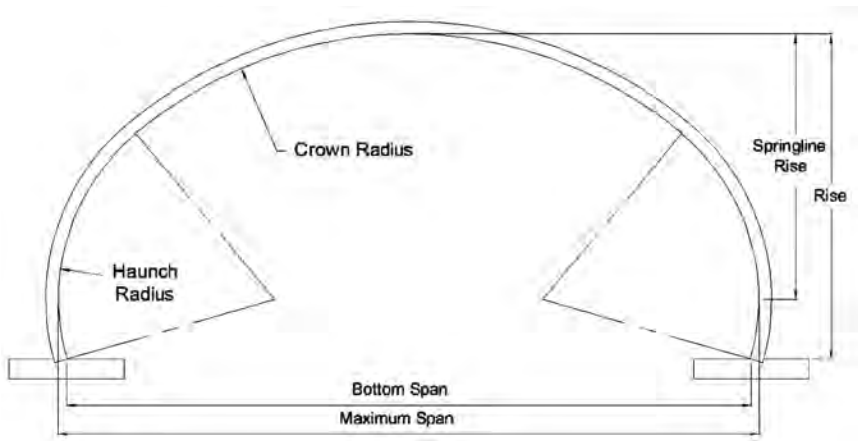
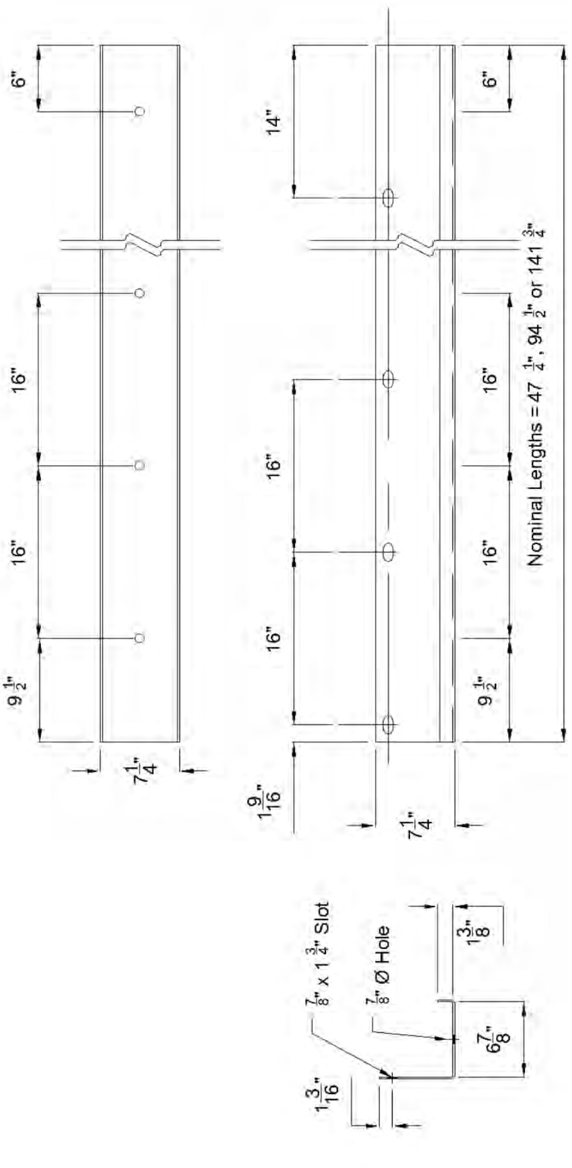


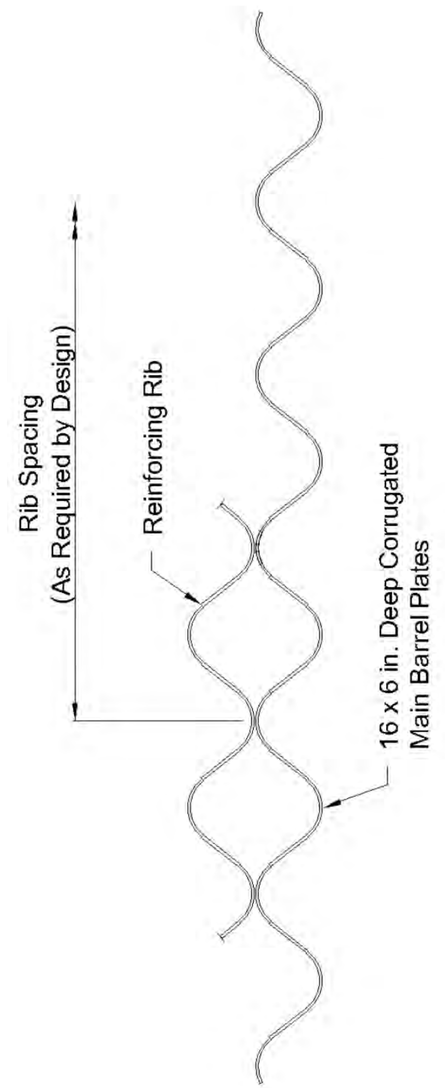
Table 2.46

Structural plate two radius arches — size and layout details
16 x 6 in. corrugations — bolted seams

Maximum Span ft-in.	Rise ft-in.	Bottom Span ft-in.	End Area ft^2	Crown H	Crown Radius in.	Side H	Side Radius in.
20 - 4	9 - 2	20 - 3	149.4	12	142	5	87
21 - 7	9 - 4	21 - 6	162.7	13	154	5	87
22 - 10	9 - 7	22 - 9	176.4	14	165	5	87
25 - 5	10 - 1	25 - 3	205.0	16	189	5	87
26 - 8	10 - 4	26 - 7	219.9	17	201	5	87
27 - 7	11 - 7	27 - 5	256.9	17	201	6	102
27 - 11	10 - 7	27 - 10	235.2	18	213	5	87
29 - 10	13 - 1	29 - 7	314.3	18	213	7	118
29 - 3	10 - 10	29 - 1	251.0	19	224	5	87
30 - 6	11 - 1	30 - 5	267.2	20	236	5	87
32 - 8	12 - 6	32 - 7	327.2	21	248	6	102
33 - 12	12 - 9	33 - 10	345.8	22	260	6	102
34 - 5	11 - 9	34 - 4	317.7	23	274	5	87
36 - 4	14 - 3	36 - 1	412.9	23	274	7	118
35 - 8	11 - 11	35 - 7	335.5	24	285	5	87
37 - 7	14 - 5	37 - 5	433.9	24	285	7	118
36 - 12	12 - 2	36 - 10	353.7	25	297	5	87
38 - 10	14 - 8	38 - 8	455.3	25	297	7	118
40 - 1	14 - 11	39 - 11	477.2	26	309	7	118
42 - 7	16 - 5	42 - 5	556.3	27	321	8	138
43 - 10	16 - 7	43 - 8	580.5	28	333	8	138
45 - 1	16 - 10	44 - 11	605.2	29	344	8	138
46 - 5	17 - 1	46 - 3	630.3	30	356	8	138
48 - 7	18 - 7	48 - 5	720.8	31	368	9	154
49 - 11	18 - 10	49 - 8	748.3	32	380	9	154
51 - 2	19 - 1	50 - 11	776.2	33	392	9	154
53 - 0	21 - 7	52 - 9	917.9	33	392	11	185



■ **Figure 2.16** General dimensions of unbalanced channel for 16 x 6 in. deep corrugated structural plate arches.



■ **Figure 2.17** Reinforcing rib details for deep corrugated structural plate, 16 x 6 in. corrugation profile.

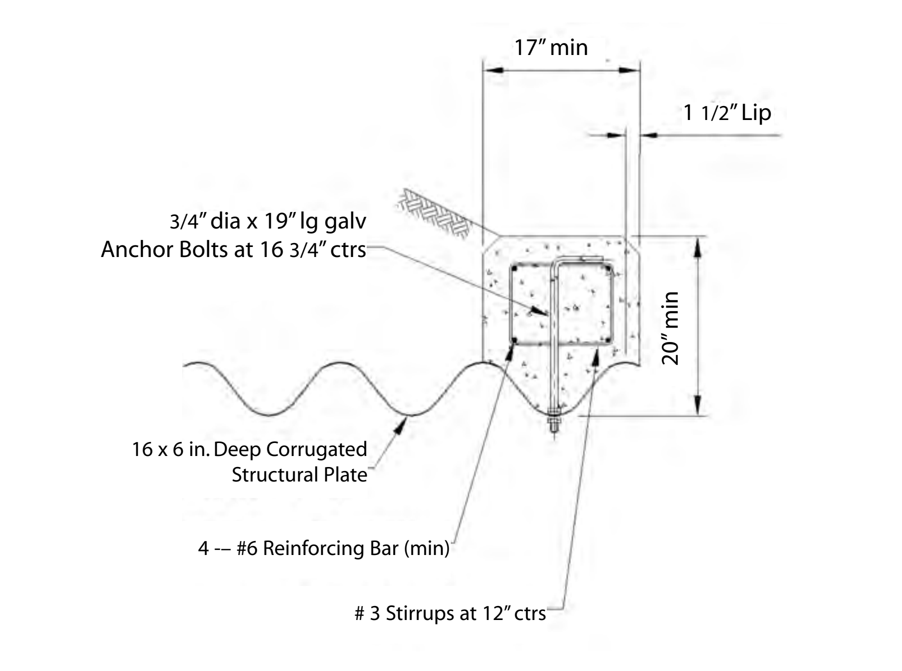


Figure 2.18 Typical reinforced concrete collar detail for 16 x 6 in. deep corrugated structural plate.

CORRUGATED STEEL BOX CULVERTS

Corrugated steel box culverts closely resemble the rectangular shape of a low, wide box. These bridges or culverts are manufactured from standard structural plate or deep corrugated structural plate (see preceding section). This is made possible by the addition of special reinforcing elements to standard structural plate or the addition of special rib plates (where required) to the standard and deep corrugated structural plate. The resulting combined section develops the flexural capacity required for the very flat top and sharp corners.

The foundation for box culverts can be designed as a conventional concrete footing, with steel footer pads (as shown in Figure 2.13), or a full steel invert.

Corrugated steel box culverts can be designed for low, wide waterway requirements with heights of cover between 1.4 to 5.0 feet (measured from the outside crest of main barrel) and various loading situations. Box culverts are available in standard spans of 9 feet 2 inches to 53 feet 0 inches and rises of 2 feet 6 inches to 13 feet 1 inch. Tables 2.47, 2.48 and 2.49 provides representative sizes available. Contact your CSP fabricator for information on box culvert sizes not listed in Tables 2.47, 2.48 and 2.49.

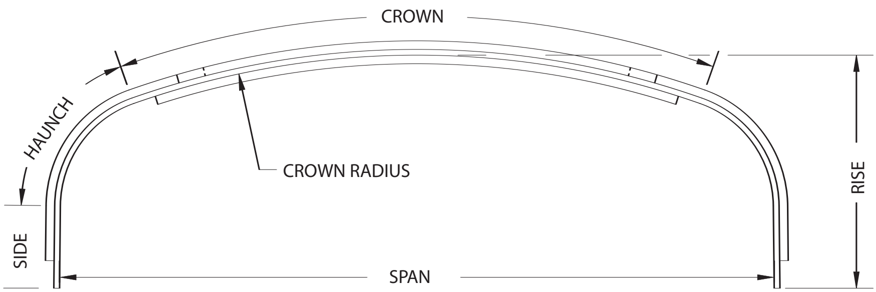


Table 2.47

Low profile box culvert — size and end area
6 x 2 in. corrugations — bolted seams

Nominal Size		Waterway	Minimum	Nominal Size		Waterway	Minimum
Span ft-in.	Rise ft-in.	Area ft^2	Cover ft	Span ft-in.	Rise ft-in.	Area ft^2	Cover ft
9-8	2-7	20.2	1.33	12-6	2-11	30.6	1.33
10-1	3-4	27.9	1.33	12-10	3-9	40.6	1.33
10-7	4-2	35.9	1.33	13-2	4-6	50.8	1.33
11-0	4-11	44.2	1.33	13-6	5-4	61.4	1.40
11-5	5-8	53.0	1.33	13-10	6-1	72.2	1.40
11-10	6-5	61.9	1.33	14-2	6-10	83.1	1.40
12-3	7-3	71.2	1.33	14-6	7-8	94.4	1.50
12-8	8-0	80.9	1.33	14-10	8-5	105.99	1.50
10-5	2-8	23.1	1.33	13-3	3-1	33.88	1.33
10-10	3-5	31.2	1.33	13-6	3-10	44.3	1.40
11-2	4-3	39.7	1.33	13-10	4-8	55.1	1.40
11-7	5-0	48.4	1.33	14-1	5-5	66.0	1.40
12-0	5-9	57.7	1.33	14-5	6-2	77.22	1.40
12-5	6-7	67.0	1.33	14-9	7-0	88.8	1.50
12-10	7-4	76.9	1.33	15-0	7-9	100.55	1.50
13-3	8-2	85.1	1.33	15-4	8-7	110.1	1.50
11-1	2-9	25.1	1.33	13-11	3-2	36.6	1.40
11-6	3-6	34.0	1.33	14-2	3-11	47.88	1.40
11-10	4-4	43.1	1.33	14-6	4-9	59.1	1.50
12-3	5-1	52.5	1.33	14-9	5-6	70.7	1.50
12-7	5-10	62.3	1.33	15-00	6-4	82.4	1.50
13-00	6-8	72.3	1.33	15-4	7-1	94.5	1.50
13-5	7-5	82.6	1.33	15-7	7-11	106.8	1.60
13-9	8-2	93.2	1.40	15-10	8-8	119.2	1.60
11-10	2-10	28.0	1.33	14-7	3-3	40.2	1.50
12-2	3-8	37.4	1.33	14-10	4-1	51.8	1.50
12-6	4-5	47.0	1.33	15-1	4-10	63.6	1.50
12-10	5-2	57.0	1.33	15-4	5-8	75.6	1.50
13-3	6-0	67.1	1.33	15-7	6-5	88.0	1.60
13-7	6-9	77.6	1.40	15-10	7-3	100.3	1.60
13-11	7-6	88.2	1.40	16-1	8-0	113.0	1.60
14-3	8-4	97.5	1.40	16-4	8-10	123.0	1.60
15-3	3-5	43.3	1.50	18-0	3-11	58.8	1.80
15-6	4-2	55.9	1.60	18-1	4-9	73.3	1.80
15-9	5-0	68.1	1.60	18-3	5-7	87.9	1.80
16-0	5-9	80.7	1.60	18-4	6-4	102.66	1.80
16-2	6-7	93.2	1.60	18-6	7-2	117.3	1.90
16-5	7-4	106.5	1.70	18-7	7-11	132.3	1.90
16-8	8-2	119.7	1.70	18-9	8-9	147.3	1.90
16-10	8-11	133.0	1.60	18-10	9-6	162.4	1.90

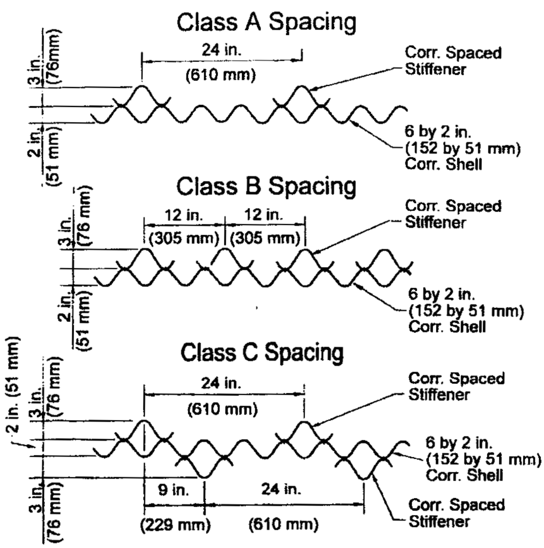
Notes:
1. Maximum cover is 5 ft. Where cover in excess of 5 ft is required, consult with manufacturer.
2. To determine minimum allowable cover, add 3 in. to rise dimension to allow for material thickness
3. If interior ribs are used, reduce waterway area by 5%.

Table 2.47 *continued*

Low profile box culvert — size and end area
6 x 2 in. corrugations — bolted seams

Nominal Size		Waterway	Minimum	Nominal Size		Waterway	Minimum
Span ft-in.	Rise ft-in.	Area ft^2	Cover ft	Span ft-in.	Rise ft-in.	Area ft^2	Cover ft
16-0	3-6	47.1	1.60	18-8	4-1	63.4	1.90
16-2	4-4	59.9	1.60	18-9	4-11	78.4	1.90
16-4	5-1	72.8	1.70	18-10	5-8	93.4	1.90
16-7	5-11	85.8	1.70	18-11	6-6	108.5	1.90
16-9	6-9	99.1	1.70	19-1	7-4	123.6	1.90
17-0	7-6	112.4	1.70	19-2	8-1	138.9	1.90
17-2	8-4	126.0	1.70	19-3	8-11	154.2	1.90
17-5	9-1	137.0	1.70	19-4	9-8	166.0	1.90
16-8	3-8	50.7	1.70	19-4	4-3	67.7	1.90
16-10	4-6	64.1	1.70	19-5	5-1	83.3	1.90
17-0	5-3	77.6	1.70	19-6	5-10	98.9	2.00
17-2	6-1	91.3	1.70	19-6	6-8	114.6	2.00
17-4	6-10	105.1	1.70	19-7	7-5	126.6	2.00
17-6	7-8	119.1	1.80	19-8	8-3	146.2	2.00
17-8	8-5	133.2	1.80	19-9	9-1	162.0	2.00
17-10	9-3	147.4	1.80	19-10	9-10	178.0	2.00
17-14	3-10	55.0	1.70	20-8	4-7	77.5	2.10
17-6	4-7	68.8	1.80	20-8	5-5	94.1	2.10
17-7	5-5	82.8	1.80	20-8	6-2	110.7	2.10
17-9	6-2	96.8	1.80	20-8	7-0	127.4	2.10
17-11	7-0	111.1	1.80	20-9	7-10	143.3	2.10
18-1	7-9	125.4	1.80	20-9	8-7	160.7	2.10
18-3	8-7	139.8	1.80	20-9	9-5	177.4	2.10
18-5	9-4	151.2	1.80	20-9	10-2	194.2	2.10

Notes: 1. Maximum cover is 5 ft. Where cover in excess of 5 ft is required, consult with manufacturer.
2. To determine minimum allowable cover, add 3 in. to rise dimension to allow for material thickness
3. If interior ribs are used, reduce waterway area by 5%.



■ **Figure 2.19** Reinforcing rib details for 6 x 2 in. structural plate box.

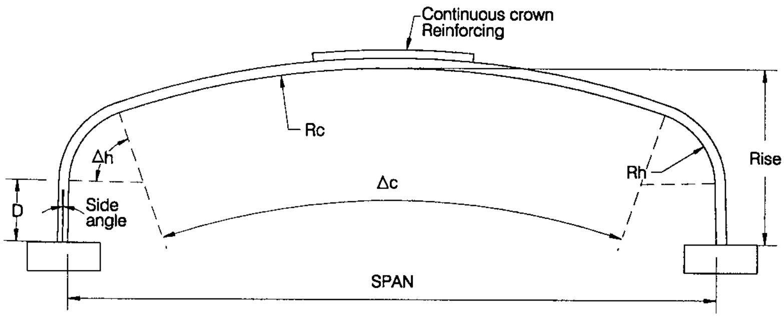


Table 2.48

DCSP Type I — size and layout details for box culverts
15 x 5 1/2 in corrugation profile — bolted seams

Span ft-in.	Rise ft-in.	Area ft^2	Crown Radius in.	Haunch Radius in.	Side Angle degree	Total *S
10'-4 3/4"	3'-10 1/2"	33.6	347.2	40	14.00	11
11'-7 7/8"	4'-7 7/8"	46.6	347.2	40	10.00	13
12'-7 1/8"	4'-9 5/8"	53.2	347.2	40	6.00	14
13' 0"	7' 3"	79.1	347.2	40	12.66	17
12' 8 1/8"	4'-1 3/4"	45.0	347.2	40	11.35	13
13' 5 5/8"	6' 1 1/4"	70.6	347.2	40	11.35	16
13'-9 3/4"	4'-3 1/2"	51.2	347.2	40	10.04	14
15'-6 1/2"	6'-5 1/4"	87.8	347.2	40	8.73	18
14'-11 1/4"	4'-5 1/2"	57.7	347.2	40	8.73	15
16'-0 1/2"	5'-3 1/2"	75.0	347.2	40	4.76	17
15'-11 1/4"	7'-9"	108.6	347.2	40	8.73	20
16'-10 7/8"	7'-11 1/4"	119.0	347.2	40	7.42	21
17'-1 3/8"	5'-5 3/4"	83.1	347.2	40	3.45	18
17'-7"	6'-9 3/4"	106.5	347.2	40	6.11	20
17'-5 1/2"	4'-8 7/8"	71.3	347.2	40	8.53	17
17'-10 3/8"	8'-1 5/8"	129.9	347.2	40	6.11	22
18'-6 3/4"	4'-11 1/4"	78.9	347.2	40	7.22	18
19'-6 3/8"	8'-8 1/8"	153.2	347.2	40	3.81	24
19'-4"	5'-2 3/4"	87.9	347.2	40	3.49	19
20'-2 5/8"	6'-2 7/8"	111.2	347.2	40	2.50	21
20'-5 1/2"	8'-10 7/8"	165.3	347.2	40	2.50	25
20'-8 7/8"	5'-4 3/4"	95.9	347.2	40	4.60	20
21'-3 1/8"	6'-5 3/4"	121.1	347.2	40	1.19	22
21'-3 3/4"	7'-9 3/4"	149.5	347.2	40	1.19	24
21'-9 5/8"	5'-7 5/8"	105.2	347.2	40	3.29	21
22'-10 3/8"	5'-10 3/4"	114.8	347.2	40	1.98	22
22'-11 1/2"	7'-2 5/8"	145.3	347.2	40	1.98	24
23'-0 5/8"	8'-6 3/4"	176.0	347.2	40	1.98	26
23' 11"	6'-1 7/8"	125.1	347.2	40	0.67	23
23'-11 3/8"	7'-5 7/8"	156.9	347.2	40	0.67	25
23'-11 3/4"	8'-9 7/8"	189.0	347.2	40	0.67	27

*S = 16 in.

Note:
1. All dimensions are to the inside crest and subject to manufacturing tolerances.
2. Other sizes are available.
3. All structures should be reviewed based on live load and geotechnical condition.

Table 2.48 *continued*

DCSP Type I — size and layout details for box culverts
15 x 5 1/2 in corrugation profile — bolted seams

Span ft-in.	Rise ft-in.	Area ft^2	Crown Radius in.	Haunch Radius in.	Side Angle degree	Total *S
24'-0 1/8"	10'-1 7/8"	220.9	347.2	40	0.67	29
24'-3 1/2"	5'-6 1/8"	109.9	347.2	40	11.38	22
25'-7 1/8"	6'-5 3/8"	136.8	347.2	40	10.07	24
26'-0 3/4"	7'-9 1/4"	170.8	347.2	40	10.07	26
28'-1 5/8"	6'-3 5/8"	149.6	450.0	40	2.09	26
28'-2 3/4"	7'-7 5/8"	187.1	450.0	40	2.09	28
28'-4"	8'-11 5/8"	224.9	450.0	40	2.09	30
30'-0"	6'-4 3/8"	157.6	450.0	40	5.94	27
30'-3 1/4"	7'-8 1/4"	197.5	450.0	40	5.94	29
30'-6 1/2"	9'-0 1/4"	237.9	450.0	40	5.94	31
32'-2 1/8"	6'-11"	182.1	450.0	40	3.92	29
32'-4 1/4"	8'-2 7/8"	225.0	450.0	40	3.92	31
32'-6 1/2"	9'-6 7/8"	268.2	450.0	40	3.92	33
34'-3 7/8"	7'-6"	209.1	450.0	40	1.89	31
34'-4 7/8"	8'-10"	254.9	450.0	40	1.89	33
34'-5 7/8"	10'-2"	300.8	450.0	40	1.89	35
35'-8 7/8"	7'-8 3/4"	221.6	450.0	40	3.32	32
35'-10 3/4"	9'-0 3/4"	269.3	450.0	40	3.32	34
36'-0 1/2"	10'-4 3/4"	317.2	450.0	40	3.32	36
38'-2 1/2"	8'-3 5/8"	250.9	450.0	40	3.71	34
38'-4 5/8"	9'-7 5/8"	301.8	450.0	40	3.71	36
38'-6 3/4"	10'-11 1/2"	353.2	450.0	40	3.71	38
40'-3"	9'-0"	284.8	450.0	40	1.68	36
40'-4"	10'-4"	338.5	450.0	40	1.68	38
40'-4 3/4"	11'-8"	392.3	450.0	40	1.68	40
42' 9"	9' 3"	330.7	646.9	57.2	1.56	39
42' 10"	10' 7"	387.7	646.9	57.2	1.56	41
46' 3"	10' 0"	383.3	646.9	57.2	1.28	42
46' 4"	11' 5"	445.0	646.9	57.2	1.28	44
49' 3"	10' 5"	413.1	646.9	57.2	1.25	44
49' 4"	11' 9"	478.8	646.9	57.2	1.25	46
51' 1'	12' 7"	532.4	646.9	57.2	1.25	48
51' 8"	13' 1"	561.0	646.9	57.2	1.25	50

*S = 16 in.
Note: 1. All dimensions are to the inside crest and subject to manufacturing tolerances.
2. Other sizes are available.
3. All structures should be reviewed based on live load and geotechnical condition.

36 foot span by 10 foot 5 inch rise deep corrugated box culvert with beveled ends, used for a Fish Passage project in the Willamette National Forest in Oregon.

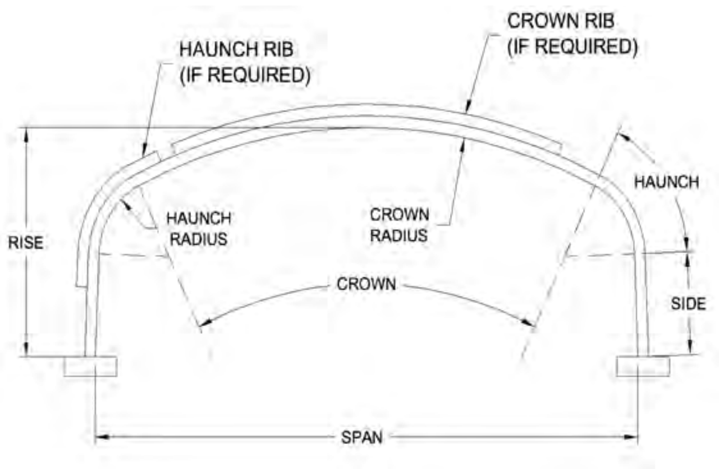


Table 2.49

DCSP Type II — size and layout details
16 x 6 in. corrugations — bolted seams

Span ft-in.	Rise ft-in.	End Area ft^2	Crown H*	Crown Radius in.	Haunch H*	Haunch Radius in.	Side H*	Side Radius in.
13 - 1	4 - 0	42.4	5.2	160	3.2	49	0.2	160
13 - 3	4 - 7	50.7	5.2	169	3.7	49	0.2	169
14 - 9	4 - 7	56.9	6	233	3.8	49	0.2	233
16 - 5	4 - 7	63.0	7.2	273	3.7	49	0.2	273
18 - 1	4 - 7	62.2	9	184	2.8	49	0.2	184
12 - 6	4 - 11	51.8	4.4	162	3.8	49	0.5	162
14 - 1	4 - 11	58.6	5.6	203	3.8	49	0.4	203
15 - 9	4 - 11	65.0	7	225	3.7	49	0.3	225
13 - 5	5 - 3	59.7	5.1	180	3.7	49	0.75	180
17 - 1	5 - 3	73.6	8.7	195	3.3	49	0.35	195
20 - 0	5 - 3	52.2	10.4	286	3.6	49	0.2	286
15 - 1	5 - 11	77.1	6.2	230	3.8	49	1.1	230
17 - 1	5 - 11	85.4	8.2	215	3.4	49	1	215
19 - 0	5 - 11	92.8	10.3	214	3.2	49	0.65	214
22 - 4	5 - 11	107.6	12.7	270	3.3	49	0.35	270
13 - 11	6 - 7	78.7	5.1	190	3.5	49	1.95	190
16 - 1	6 - 7	88.4	7.5	171	2.9	49	1.85	171
18 - 1	6 - 7	96.9	10.6	161	2.7	49	1	161
20 - 0	6 - 7	102.9	12.6	175	2.7	49	0.5	175
21 - 4	6 - 7	113.7	12.8	213	3	49	0.6	213
23 - 11	6 - 7	130.9	14	295	3.5	49	0.5	295
14 - 9	7 - 3	89.3	5.2	181	3	49	2.9	181
16 - 5	7 - 3	99.6	7.9	163	2.7	49	2.35	163
17 - 1	7 - 3	108.5	7.4	280	3.7	49	2.1	280
22 - 12	7 - 3	137.7	13.5	252	3.1	49	1.15	252
24 - 11	7 - 3	146.5	15.3	263	3	49	0.85	263
16 - 1	7 - 10	110.0	6.3	238	3.4	49	2.95	238
18 - 1	7 - 10	121.7	8.5	213	3	49	2.75	213
20 - 0	7 - 10	132.7	10.8	209	2.8	49	2.3	209
21 - 8	7 - 10	142.8	12.8	218	2.9	49	1.7	218
23 - 4	7 - 10	153.2	13.9	246	3	49	1.55	246

*H = 16.75 in.
Notes: 1. All dimensions are to the inside crest and are subject to manufacturing tolerances.
2. Sizes are representative, other sizes may be available, contact your manufacturer.

Table 2.49 *continued*

DCSP Type II — size and layout details
16 x 6 in. corrugations — bolted seams

Span ft-in.	Rise ft-in.	End Area ft^2	Crown H*	Crown Radius in.	Haunch H*	Haunch Radius in.	Side H*	Side Radius in.
25 - 5	7 - 10	163.3	15.9	254	2.8	49	1.25	254
19 - 8	8 - 6	145.8	9.4	264	3.2	49	3.1	264
22 - 12	8 - 6	169.3	12.5	292	3.2	49	2.55	292
23 - 11	8 - 6	169.3	14.8	231	2.7	49	1.9	231
20 - 0	9 - 2	159.2	9.5	262	3.1	49	3.65	262
21 - 12	9 - 2	173.4	10.7	291	3	49	3.65	291
25 - 0	9 - 2	197.7	14.4	298	3.1	49	2.7	298
22 - 4	9 - 10	189.0	12.1	238	2.7	49	3.75	238
24 - 3	9 - 10	203.1	13	275	2.7	49	3.8	275
25 - 6	9 - 10	215.6	14.9	284	2.9	49	3.15	284
26 - 3	6 - 7	137.8	16.2	294	3.2	49	0.2	294
28 - 8	6 - 7	155.3	17.2	400	3.6	49	0.3	400
26 - 3	7 - 2	155.9	15.8	309	3.2	49	0.9	309
29 - 6	7 - 3	173.8	18.1	367	3.2	49	0.75	367
32 - 10	7 - 3	190.5	20.6	419	3.3	49	0.4	419
27 - 11	7 - 11	184.5	16.4	362	3.2	49	1.6	362
31 - 2	7 - 10	203.4	19	413	3.3	49	1.2	413
36 - 1	7 - 11	231.0	23	477	3.5	49	0.5	477
27 - 11	8 - 6	203.1	16.5	362	3.3	49	1.95	362
32 - 10	8 - 6	221.7	21.6	337	2.8	49	0.9	337
37 - 9	8 - 6	251.0	25.2	411	3	49	0.4	411
26 - 3	9 - 2	209.9	15.1	345	3.3	49	2.65	345
32 - 10	9 - 2	244.7	21.1	350	2.8	49	1.65	350
36 - 1	9 - 2	266.6	23.5	402	2.9	49	1.35	402
39 - 4	9 - 2	287.3	26.2	444	3.1	49	0.8	444
26 - 3	9 - 10	227.7	14.4	393	3.4	49	3.4	393
29 - 6	9 - 10	242.6	17.7	325	2.7	49	2.95	325
32 - 10	9 - 10	267.7	20.5	366	2.8	49	2.45	366
36 - 1	9 - 10	291.9	23	418	2.9	49	2.1	418
39 - 4	9 - 10	315.4	25.6	465	3	49	1.7	465
29 - 6	10 - 6	262.7	16.9	355	2.8	49	3.75	355
32 - 10	10 - 6	289.3	20.5	372	2.9	49	2.85	372
36 - 1	10 - 6	315.8	23.1	419	3	49	2.45	419
39 - 4	10 - 6	341.5	25.7	465	3.1	49	2.05	465
32 - 11	11 - 2	312.0	20.2	383	2.9	49	3.5	383
36 - 1	11 - 2	341.1	22.5	441	3	49	3.25	441
39 - 4	11 - 2	368.8	25.4	478	3.1	49	2.7	478
36 - 3	11 - 10	365.4	22.7	438	3	49	3.65	438
39 - 4	11 - 10	380.4	25.8	410	2.6	49	3	410
37 - 9	12 - 5	391.2	24.3	394	2.6	49	3.75	394
41 - 0	12 - 5	422.8	26.8	443	2.7	49	3.4	443

*H = 16.75 in.
Notes: 1. All dimensions are to the inside crest and are subject to manufacturing tolerances.
2. Sizes are representative, other sizes may be available, contact your manufacturer.

SPECIFICATIONS

Specifications in Common Use

Specifications are divided into three basic classes – those covering design, materials, and installation. These classes are covered in Tables 2.50, 2.51 and 2.52.

Table 2.50

Design specifications

Agency	Reference
AASHTO	Standard Specifications for Highway Bridges—Division I, Section 12 LRFD Bridge Design Specifications – Section 12
ASTM	Standard Practice for Structural Design of Corrugated Steel Pipe, Pipe Arches, and Arches for Storm and Sanitary Sewers and Other Buried Applications—ASTM A796
AREMA	Manual for Railway Engineering – Section 4.9

Table 2.51

Material description and specifications

Material	Description	Specifications AASHTO	Specifications ASTM
Zinc Coated Sheets & Coils	Steel base metal* with 2 oz per ft^2 zinc coating	M-218	A929M
Polymer Coated Sheets and Coils	Polymer coatings applied to sheets* and coils*, 0.010 in. both sides	M-246	A742M
Aluminum Coated Coils – Type 2	Steel base metal* coated with 1 oz. per ft^2 of pure aluminum	M-274	A929M
Sewer and Drainage Pipe	Corrugated pipe fabricated from any of the above sheets or coils. Pipe is fabricated by corrugating continuous coils into helical form with lock seam or welded seam, or by rolling annular corrugated mill sheets and riveting or spot welding seams: 1. Galvanized corrugated steel pipe 2. Polymeric pre-coated sewer and drainage pipe 3. Aluminized Type 2 corrugated steel pipe 4. Structural plate pipe	 M-36 M-245 M-274 M-167	 A760M A762M A760M A761M
Asphalt Coated Steel Sewer Pipe	Corrugated steel pipe of any of the types shown above with a 0.050 in. asphalt coating	M-190	A849
Invert Paved Steel Sewer Pipe	Corrugated steel pipe of any one of the types shown above: a. Asphalt coated pipe with 0.050 in. asphalt coating and pavement poured in the invert to cover the corrugation by 1/8 in. b. With a field applied 3 in. (3250 psi) concrete invert or 1 1/2 in. high strength (9600 psi) concrete invert. c. With polymer material applied 0.050 in. above the crest in the invert.	 M-190	 A849 A849 A849
Fully Lined Steel Sewer Pipe	Corrugated steel pipe of the types shown above: a. With an internal asphalt lining centrifugally spun in place. b. With an internal concrete lining in place. c. Corrugated steel pipe with a smooth steel liner integrally formed with the corrugated shell or. d. Corrugated steel pipe with a single thickness of smooth sheet fabricated with helical ribs projected outward or. e. With concrete pavement and linings installed in the field.	 M-190 M-36 M-36	 A849 A849 A760 A760 A979
Cold Applied Bituminous Coatings	Mastic or coal tar base coatings of various viscosities for field or shop coating of corrugated pipe or structural plate.	M-243	A849
Gaskets and Sealants	1. Standard O-ring gaskets 2. Sponge neoprene sleeve gaskets 3. Gasketing strips, butyl or neoprene 4. Mastic sealant		C443 D1056

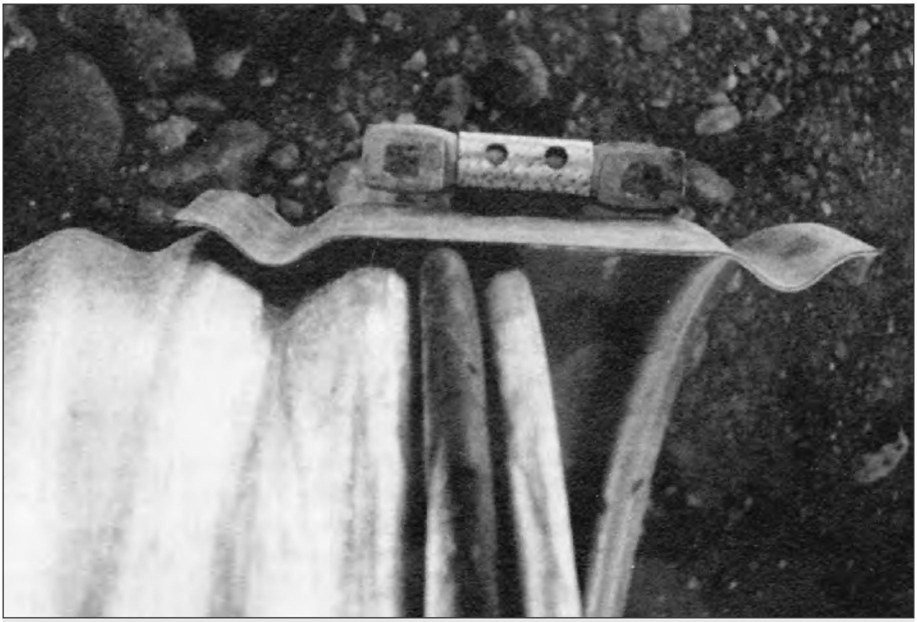
* yield point – 33 ksi min.; tensile strength – 45 ksi min.; elongation (2 in.) – 20% min.

Table 2.52

Installation specifications

Agency	Reference
AASHTO	Standard Specification for Highway Bridges-Division II, Section 26 LRFD Bridge Construction Specifications
ASTM	Standard Practice for Installing Factory Made Corrugated Steel Pipe for Sewers and Other Applications — ASTM A798 Standard Practice for Installing Corrugated Steel Structural Plate Pipe for Sewers and Other Applications — ASTM A807
AREMA	Manual for Railway Engineering – Section 4.12
U.S. Dept. of Agriculture — Natural Resources Conservation Service	Construction Specification Section 51 Paragraph 6 Service
U.S. Dept of Agriculture Forest Service	Specification for Construction of Roads and Bridges, Section 603.04 through 603.08.
Federal Lands Highway	FP92 Section 602.03, 602.05, 602.07, and 602.08

A flat gasket rolled back over itself ready to receive the next section of pipe.



An O-ring gasket in place in the valley of the last corrugation on the end of the pipe.

CORRUGATED STEEL PIPE COUPLING SYSTEMS

Field Joints for Corrugated Steel Pipe

The performance and material requirements for CSP coupling systems are scattered among several ASTM and AASHTO specifications. The two most commonly used specifications for defining CSP coupling systems are Section 26 of the AASHTO Standard Construction Specification for Highway Bridges and ASTM A760, Standard Specification for Corrugated Steel Pipe, Metallic Coated for Sewers and Drains. All CSP coupling systems involve one of the coupling bands depicted in Table 2.53 and may require a flat or o-ring gasket as also depicted in the same table. Figure 2.20 shows cross-section of assembled CSP coupling systems.

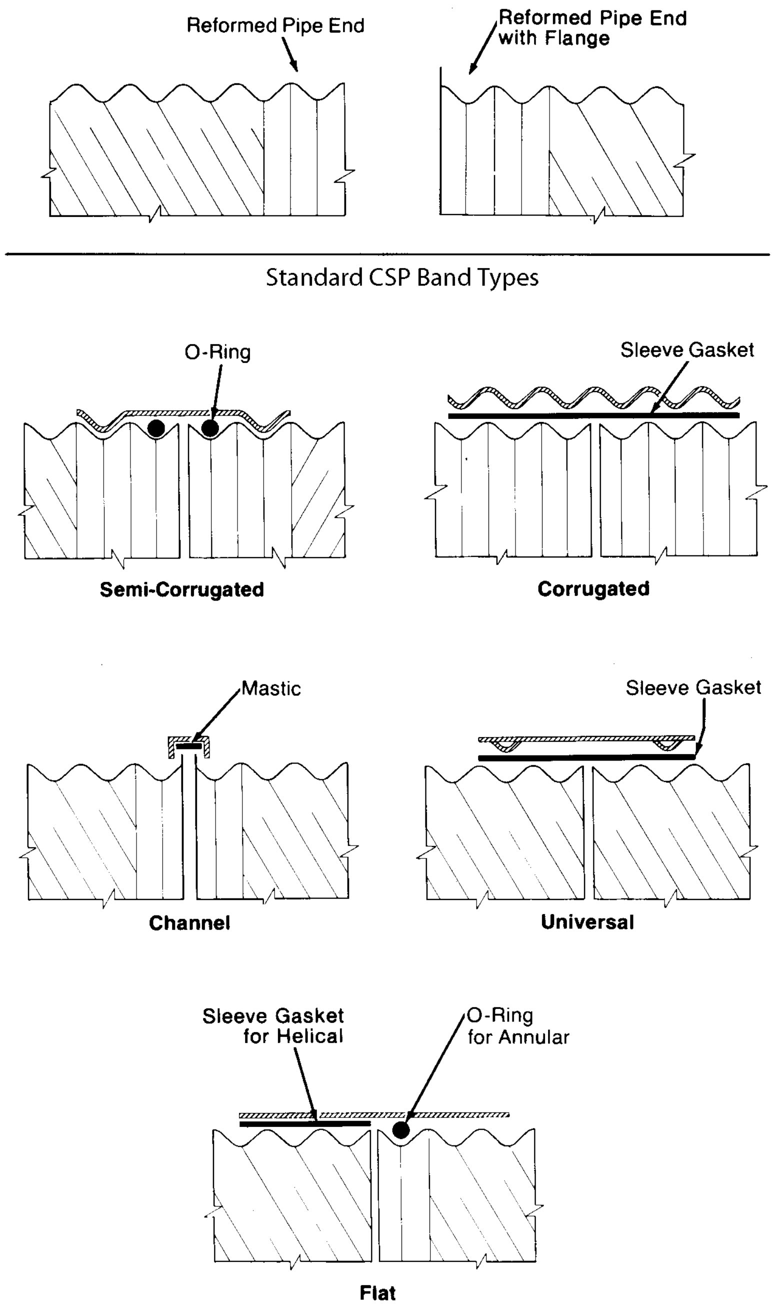
The performance of these coupling systems is defined by the amount of water or soil particles that pass through the joint. The basic joint specifications is the soil tight coupling system which has been the CSP joint specified for most culverts and storm sewers for nearly 100 years, with proven performance. The coupling system is defined by limiting the size of the openings that allow backfill materials to infiltrate into the pipe. If the specifications for a coupling system is made more restrictive, it may be necessary to include or improve a gasket as part of that system, and/or possibly include a geotextile wrap around the joint area on the outside of the pipe. Joining systems classified as leak resistant, will limit leakage to a limited amount of water passing through the joint. Consult NCSPA's website at www.ncspa.org or your local CSP fabricator for guidance on the coupling system appropriate for your project.

Table 2.53

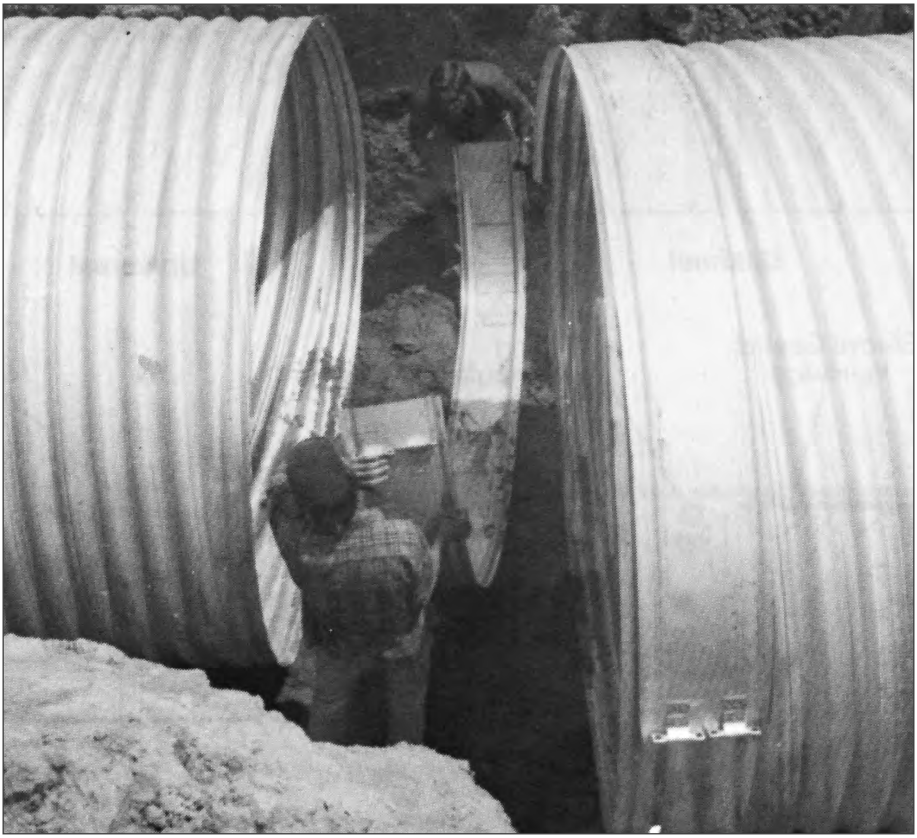
Coupling bands for corrugated steel pipe

Type Of Band	Cross Section	Angles	Bar, Bolt & Strap	Wedge Lock	Gaskets			Pipe End		
					O Ring	Sleeve or Strip	Mastic	Annular Plain	Helical Plain	Helical Reformed
Universal		X	X	X		X	X	X	X	X
Corrugated		X	X	X		X	X	X	X	X
Semi-Corrugated		X	X	X	X		X	X		X
Channel		X	X		X		X			X
Flat		X	X	X		X	X	X	X	X
Hat		X	X				X			X

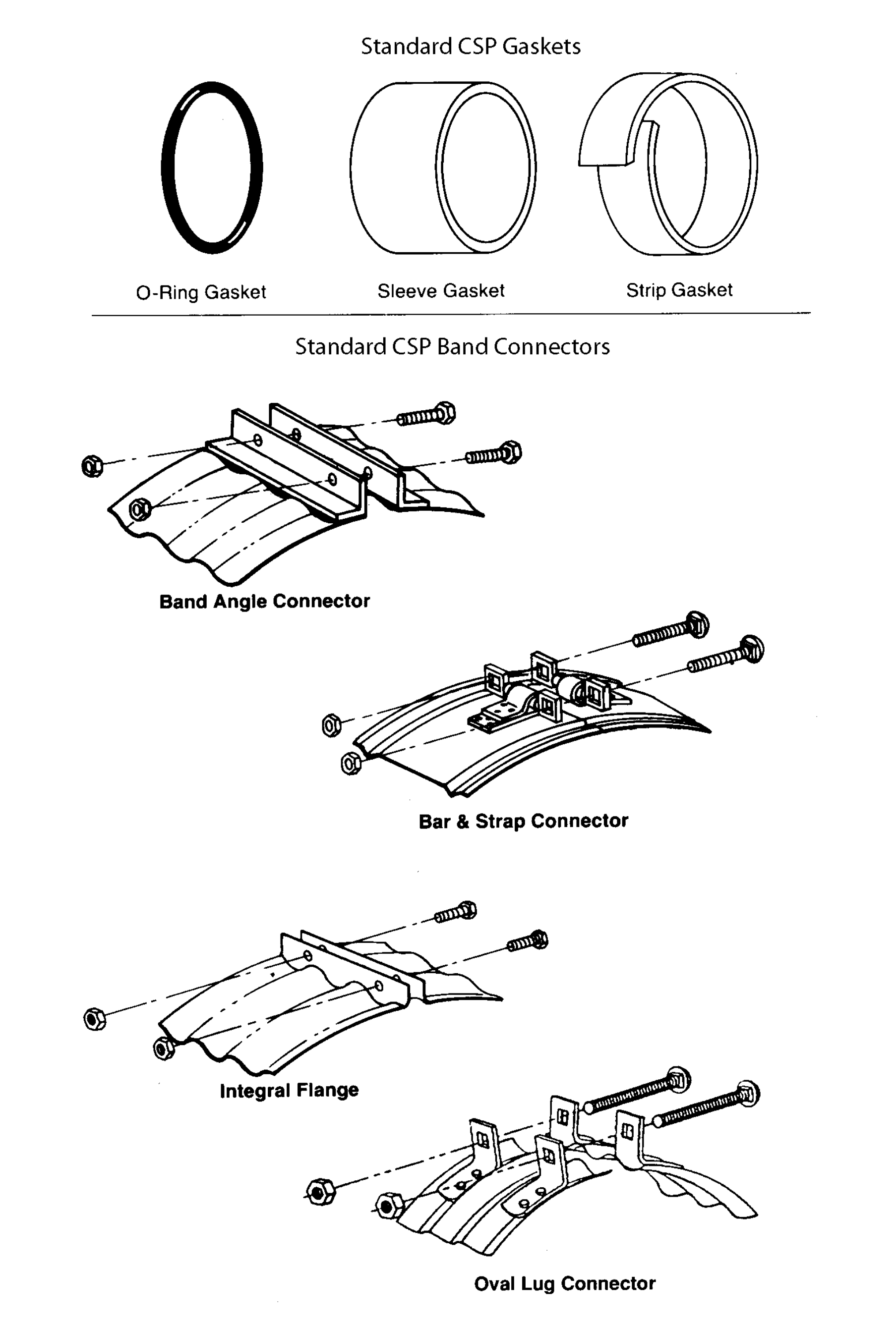
Two-piece corrugated band joins length of annular riveted pipe.



■ **Figure 2.20** Standard corrugated steel bands.



■ Two-piece bands being installed on reformed ends of CSP.

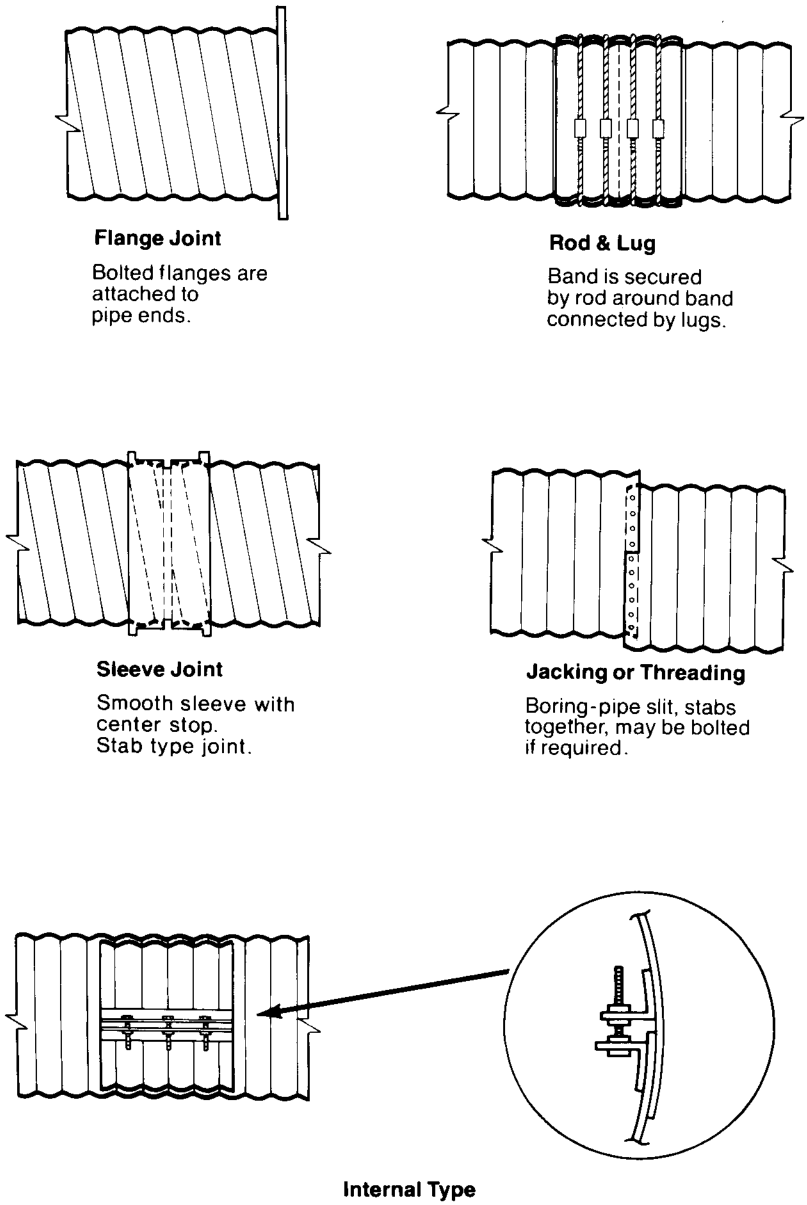


■ **Figure 2.21** Standard corrugated steel pipe band connectors and gaskets.

Corrugated steel pipe band connectors.

CSP Field Joints

For unusual conditions, such as high pressures, extreme disjointing forces, threading pipe inside existing pipe, jacking or boring pipe, and deep vertical drop inlets, a variety of special designs are available. New joints can be readily designed by manufacturers to meet particular requirements.



■ **Figure 2.22** Special connectors.

Fittings

One of the benefits of corrugated steel pipe is that it can be easily and economically fabricated into an assortment of fittings. Table 2.54 provides minimum dimensions for CSP elbows (round pipe). Table 2.55 provides minimum dimensions for CSP tees, crosses, laterals and wyes (round pipe).

Structural plate fittings are shop cut from curved corrugated plates and welded together. Theses structures are usually assembled and bolted in the shop in a trial fit to assure that all parts mate properly. The parts are then clearly marked for field assembly.

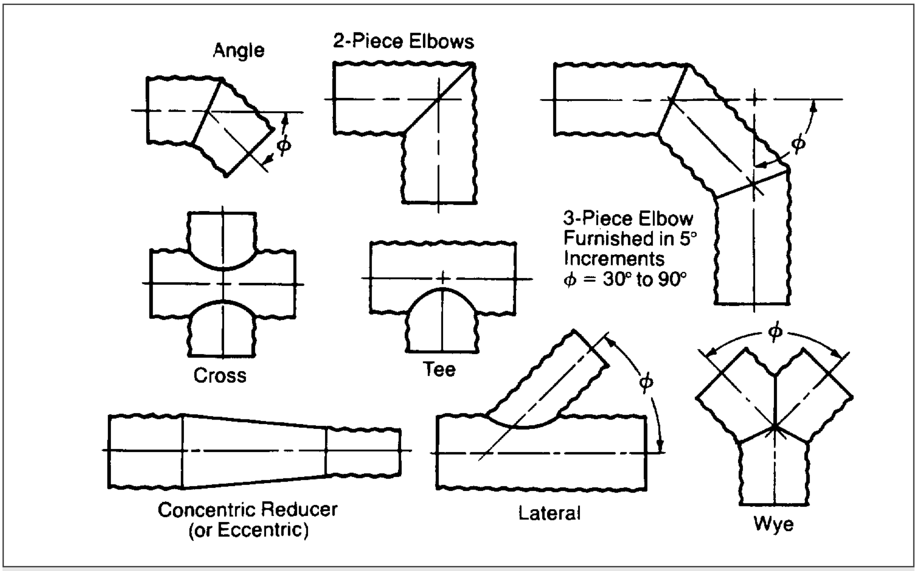
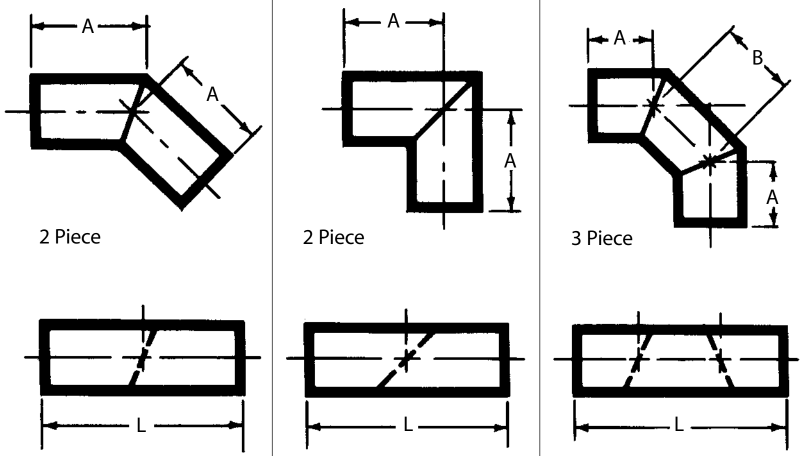


Figure 2.23 Shop fittings for corrugated steel pipe and pipe arch. Shop fabricated fittings are available for a wide variety of conditions.

Moderate horizontal curvature in a culvert or sewer can be achieved with ordinary couplings. Greater changes in alignment will require fabricated fittings.

Table 2.54

Minimum dimensions for elbows for round CSP –– all corrugations



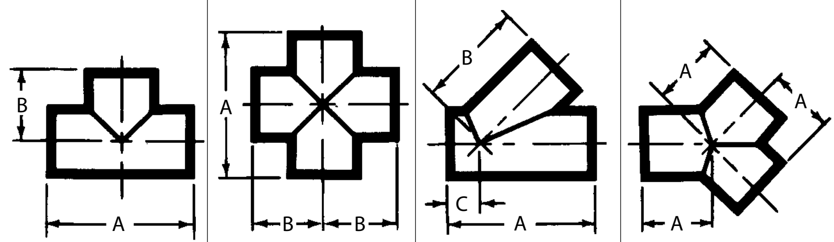
10° - 45° Elbow | 46° - 90° Elbow | 46° - 90° Elbow

Pipe Diameter (in.)	A (ft)	Total Length (ft)	Pipe Diameter (in.)	A (ft)	Total Length (ft)	Pipe Diameter (in.)	A (in.)	B (in.)	C (in.)	Total Length (ft)
6-18	1	2	6-10	1	2	6	13 1/2	8	8	2
21-48	2	4	12-27	2	4	8	14	9	7 1/2	2
54-96	3	6	30-42	3	6	10	14	10	7	2
			48-66	4	8	12	25 1/2	11	18 1/2	4
			72-84	5	10	15	26 1/2	12	18	4
			90-96	6	12	18	27	14	17	4
						21	27	15	16 1/2	4
						24	27 1/2	16	16	4
						27	27 1/2	17	15 1/2	4
						30	40	19	261/2	6
						33	40	20	26	6
						36	40 1/2	21	25 1/2	6
						42	41	23	24 1/2	6
						48	53 1/2	26	35	8
						54	54	28	34	8
						60	54 1/2	31	32 1/2	8
						66	54	33	31 1/2	8
						72	67 1/2	36	42	10
						78	68	39	40 1/2	10
						84	68 1/2	41	39 1/2	10
						90	70	46	37	10
						96	82	46	49	12

Notes: The total length (ft) and pipe diameter (in.) listed are minimum requirements for fitting fabrication. Fittings with other dimensions to satisfy specific needs are also available. All dimensions are nominal.

Table 2.55

Minimum dimensions for CSP round fittings



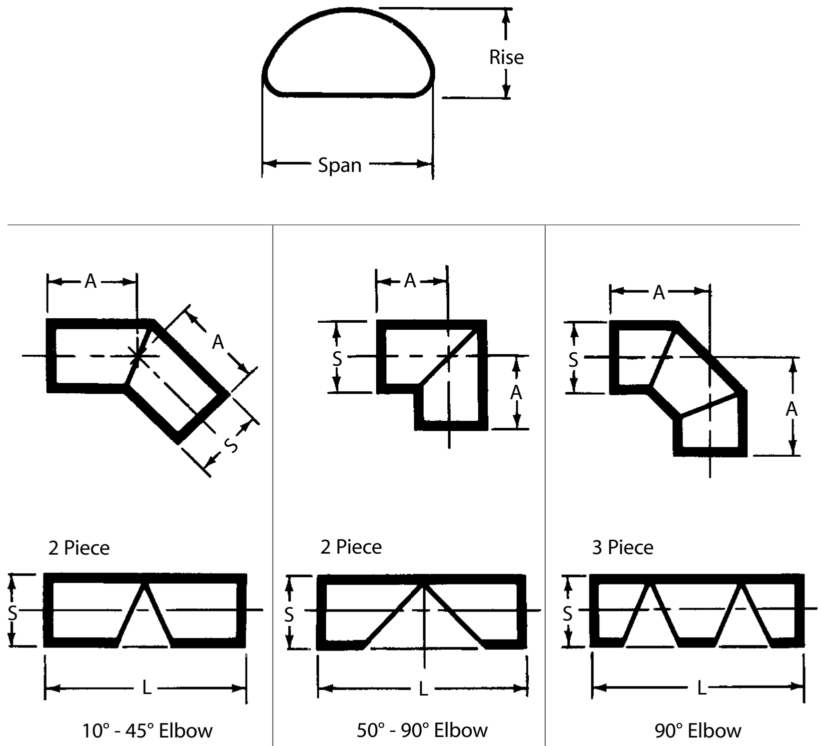
Tee | Cross | 45° Lateral | 45° Wye

Main Diam.	Stub Same or Smaller Than Main Diameter													
	Tee			Cross			45° Lateral				45° Wye			
	A	B	TL	A	B	TL	A	B	C	TL	A	B	TL	
(in.)	(ft-in.)	(ft-in.)	(ft-in.)	(ft-in.)	(ft-in.)	(ft-in.)	(ft-in.)	(ft-in.)	(ft-in.)	(ft-in.)	(ft-in.)	(ft-in.)	(ft-in.)	
6	2-6	1-3	3-9	2-6	1-3	5-0	2-9	1-6	1-2	4-3	1-1	1-3	3-7	
8	2-8	1-4	4-0	2-8	1-4	5-4	3-0	1-8	1-2	4-8	1-2	1-4	3-10	
10	2-10	1-5	4-2	2-10	1-5	5-8	3-2	1-10	1-2	5-0	1-2	1-5	4-0	
12	3-0	1-6	4-6	3-0	1-6	6-0	3-5	2-0	1-3	5-5	1-3	1-6	4-3	
15	3-3	1-8	4-11	3-3	1-8	6-6	3-9	2-3	1-3	6-0	1-3	1-8	4-7	
18	3-6	1-9	5-3	3-6	1-9	7-0	4-2	2-6	1-4	6-8	1-4	1-9	4-10	
21	3-9	1-11	5-10	3-9	1-11	7-6	4-6	2-9	1-4	7-3	1-4	1-11	5-2	
24	4-0	2-0	6-0	4-0	2-0	8-0	4-10	3-0	1-5	7-10	1-5	2-0	5-5	
27	4-3	2-2	6-5	4-3	2-2	8-6	5-2	3-3	1-6	8-5	1-5	2-2	5-9	
30	4-6	2-3	6-9	4-66	2-3	9-0	5-6	3-6	1-6	9-0	1-6	2-3	6-0	
33	4-9	2-5	7-2	4-9	2-5	9-6	5-11	3-9	1-7	9-8	1-7	2-4	6-3	
336	5-0	2-6	7-6	5-0	2-6	10-0	6-3	4-0	1-8	10-3	1-8	2-6	6-8	
42	5-6	2-9	8-3	5-6	2-9	11-0	7-0	4-6	1-9	11-6	1-9	2-9	7-3	
48	6-0	3-0	9-0	6-0	3-0	12-0	7-8	5-0	1-10	12-8	1-10	3-0	7-10	
54	6-6	3-3	9-9	-	-	-	8-4	5-6	1-11	13-10	1-11	3-3	8-5	
60	7-0	3-6	10-6	-	-	-	9-1	6-0	2-0	15-1	2-0	3-6	9-0	
66	7-6	3-9	11-3	-	-	-	9-9	6-6	2-2	16-3	2-2	3-9	9-8	
72	8-0	4-0	12-0	-	-	-	10-6	7-0	2-3	17-6	2-3	4-0	10-3	
78	8-6	4-3	12--9	-	-	-	11-2	7-6	2-4	18-8	2-4	4-3	10-10	
84	9-0	4-6	13-6	-	-	-	11-11	8-0	2-5	19-11	2-5	4-6	11-5	
90	9-6	4-9	14-3	-	-	-	12-8	8-6	2-7	21-2	2-7	4-9	12-1	
96	10-0	5-0	15-0	-	-	-	13-4	9-0	2-8	22-4	2-8	5-0	12-8	

Notes: 12 in. minimum stub dimension to allow for use of 12 in. wide connecting band.
TL - total net length needed to fabricate fitting.

Table 2.56

Minimum dimensions for CSP pipe arch fittings



Equivalent Round Diameter	Span S	Rise R	45° Elbow 2 Piece		90° Elbow 2 Piece		90° Elbow 3 Piece	
			A	L	A	L	A	L
(in.)	(in.)	(in.)	(in.)	(ft)	(in.)	(ft)	(in.)	(ft)
15	17	13	20	4	27	6	31	6
18	21	15	20	4	25	6	30	6
21	24	18	19	4	24	6	29	6
24	28	20	18	4	34	8	28	6
30	35	24	16	4	30	8	38	8
36	42	29	27	6	38	10	35	8
42	49	33	25	6	35	10	45	10
48	57	38	24	6	43	12	42	10
54	64	43	34	8	52	14	52	12
60	71	47	33	8	60	16	62	14
66	77	52	43	10	56	16	60	14
72	83	57	42	10	56	18	70	16

Notes: All dimensions are nominal.
L—length for fabrication

Saddle Branch

Saddle branches are used to connect small branch lines to the main. Saddles make it practical to tie in connections accurately after the main line is laid, or, new connections can be made effectively on old lines with saddles. Saddles can be used to connect almost any type of pipe to a CSP main. A common universal type of saddle branch stub is shown below.

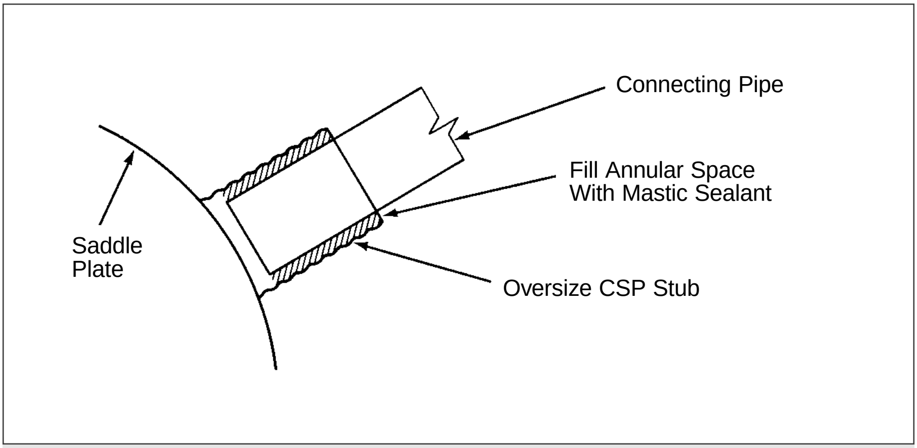


Figure 2.24 Universal connection detail using saddle branch.

Figure 2.25 Typical pre-fabricated CSP saddle branch fitting.

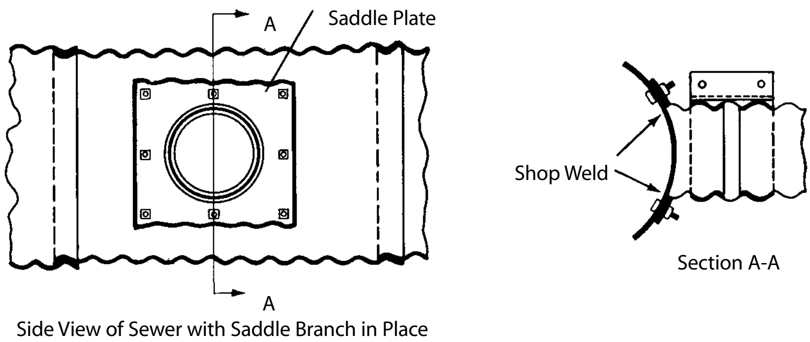
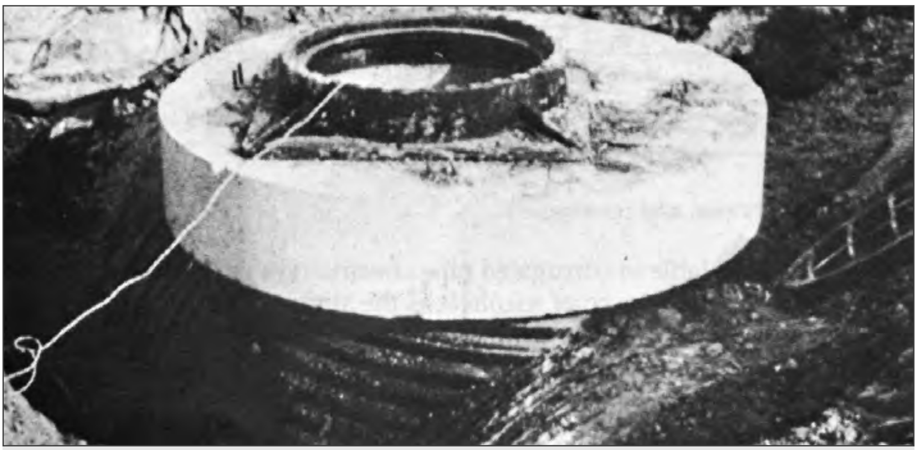


Figure 2.26 Saddle branch.

Manholes are available in corrugated steel pipe construction in two basic types as shown in Figure 2.27. The riser type of manhole is the simpler of the two and very economical. It is only feasible for trunk lines with a 36 inch diameter or greater. When junctions of smaller diameters are involved it is possible to use a vertical shaft of larger diameter CSP to connect the sewers. However, when the shaft is greater than 36 inches diameter, some reduction details must be used to suit the cover. Typical reduction details are shown. Larger sizes may require reinforcement.



Standard cast iron covers and/or steel grates are used with CSP manholes and catch basins.

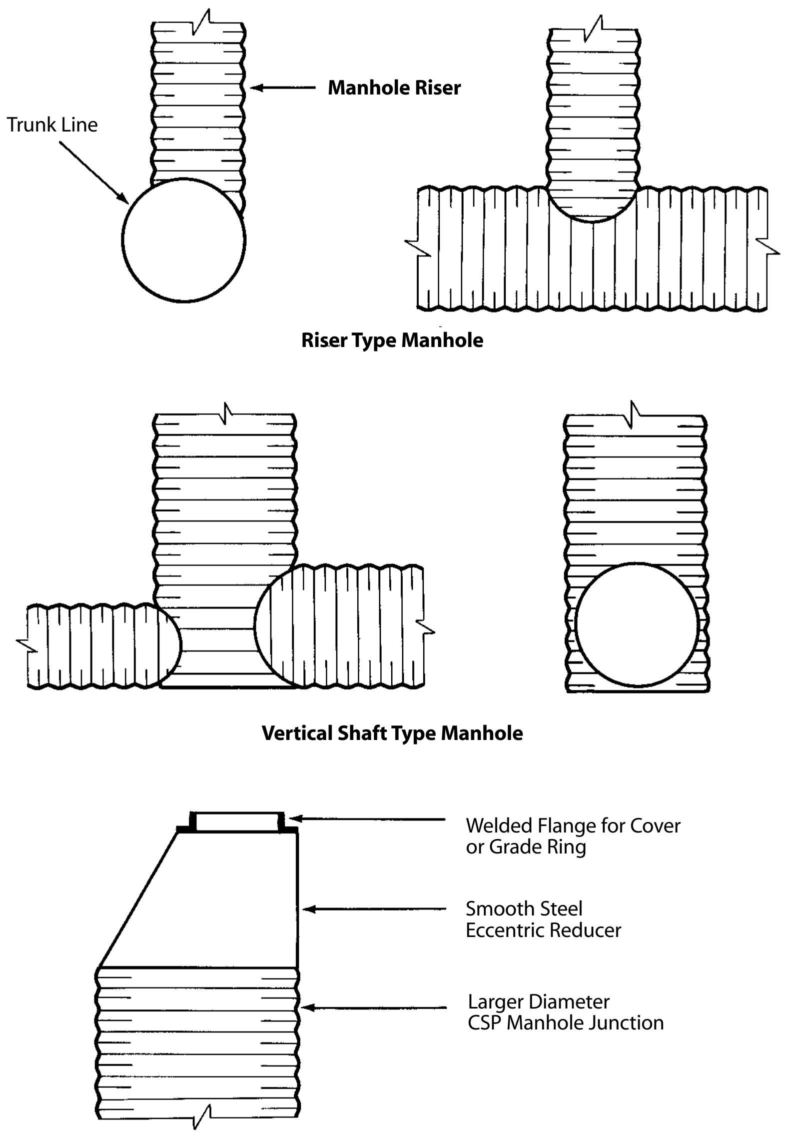
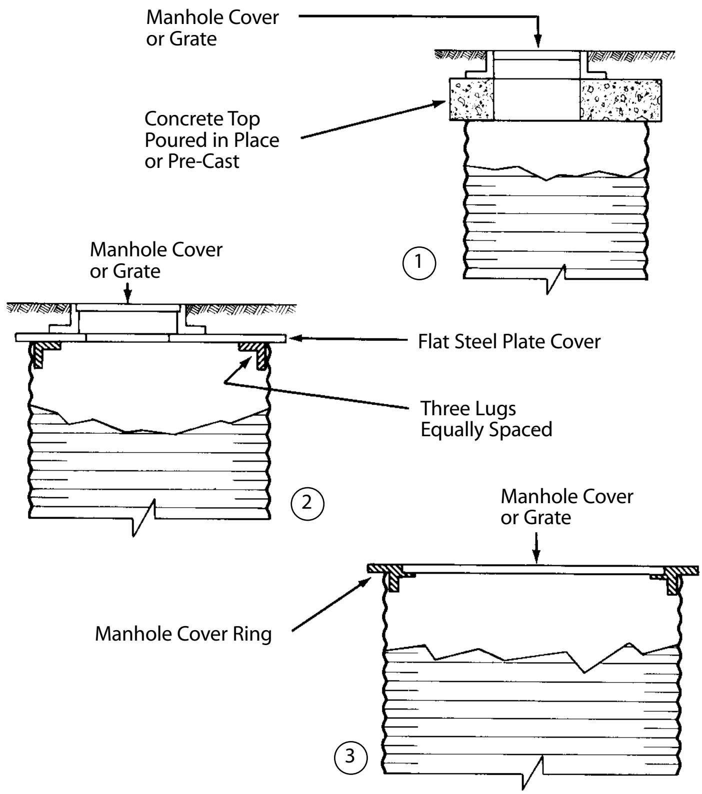


Figure 2.27 Manholes and catchbasins.



Detail (1) can be used with almost any type of surface cover or grate. Concrete grade ring may be augmented with brick to raise cover elevation in the future. Alternatively, added concrete may be poured. Direct connections of cast or fabricated plates or rings as in (2) and (3) are particularly suitable for grated inlet openings.

Figure 2.28 Manhole and catchbasin covers.

The manhole covers shown in Figure 2.28 transfer any load on the cover directly to the manhole riser. For this reason, manhole covers of this type should be placed only where vehicular traffic is not expected. If the manhole will be subjected to wheel loads, the manhole riser should be designed as per Chapter 8 of this manual.

Special galvanized steel fitting for lake water intake of power station. Sealant ribbons were used on all seams. Divers made under water bolted connection between sections.

King-size wye or lateral for large storm sewer was shop-assembled, then dismantled and shipped to the job site for final erection.

Structural plate fittings are shop cut from curved corrugated plates and bolted or welded together. Such structures are usually assembled and bolted in the shop in a trial fit to assure that all parts mate properly, then are marked clearly for field assembly.

END FINISH

Purposes

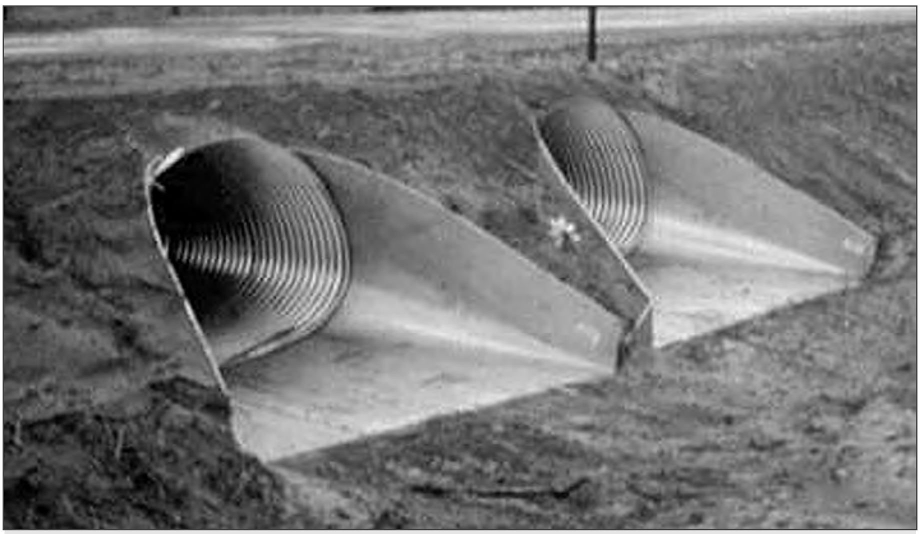
The principal purpose of end finish on corrugated steel pipe culverts or spillways is hydraulic efficiency—to prevent scour at the inlet, undermining at the outlet and to increase capacity. Other purposes may be to retain the fill slope, discourage burrowing rodents, or improve safety. For additional information, see Chapters 4 and 5, on Hydraulic Design, and Chapters 7 and 8, Structural Design and Special Design.

Types of Finish

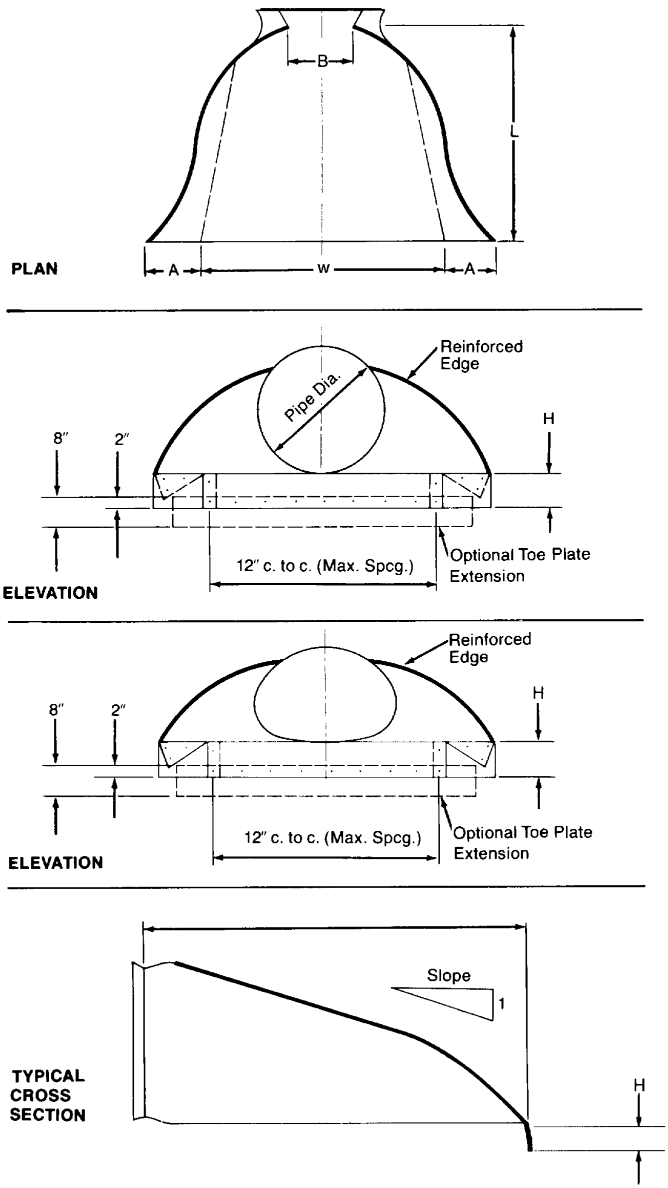
Types of steel end finishes include (1) end sections, flared and prefabricated, (2) safety slope end sections, (3) riprap and others, (4) skews and bevels, and (5) steel sheeting to serve as a low headwall and cutoff wall.

1. End Sections. Steel end sections are shop fabricated for assembly in the field by attachment to corrugated steel culverts from 6 to 96 inches diameter or pipe arches from 17 x 13 inches to 142 x 91 inches Dimensions and other data are given in Tables 2.57 to 2.59 and Figures 2.29 and 2.30.

   These end sections are listed in standard specifications of state highway departments, county road departments, railroads and others. They meet the requirements for efficient and attractive end finish on culverts, conduits, spillways and sewer outfalls. They attach to the culvert ends by simple bolted connections of various designs and can be completely salvaged if lengthening or relocation is necessary.



■ Arch flared end sections.



■ **Figure 2.29** End sections for round and pipe arch shapes.

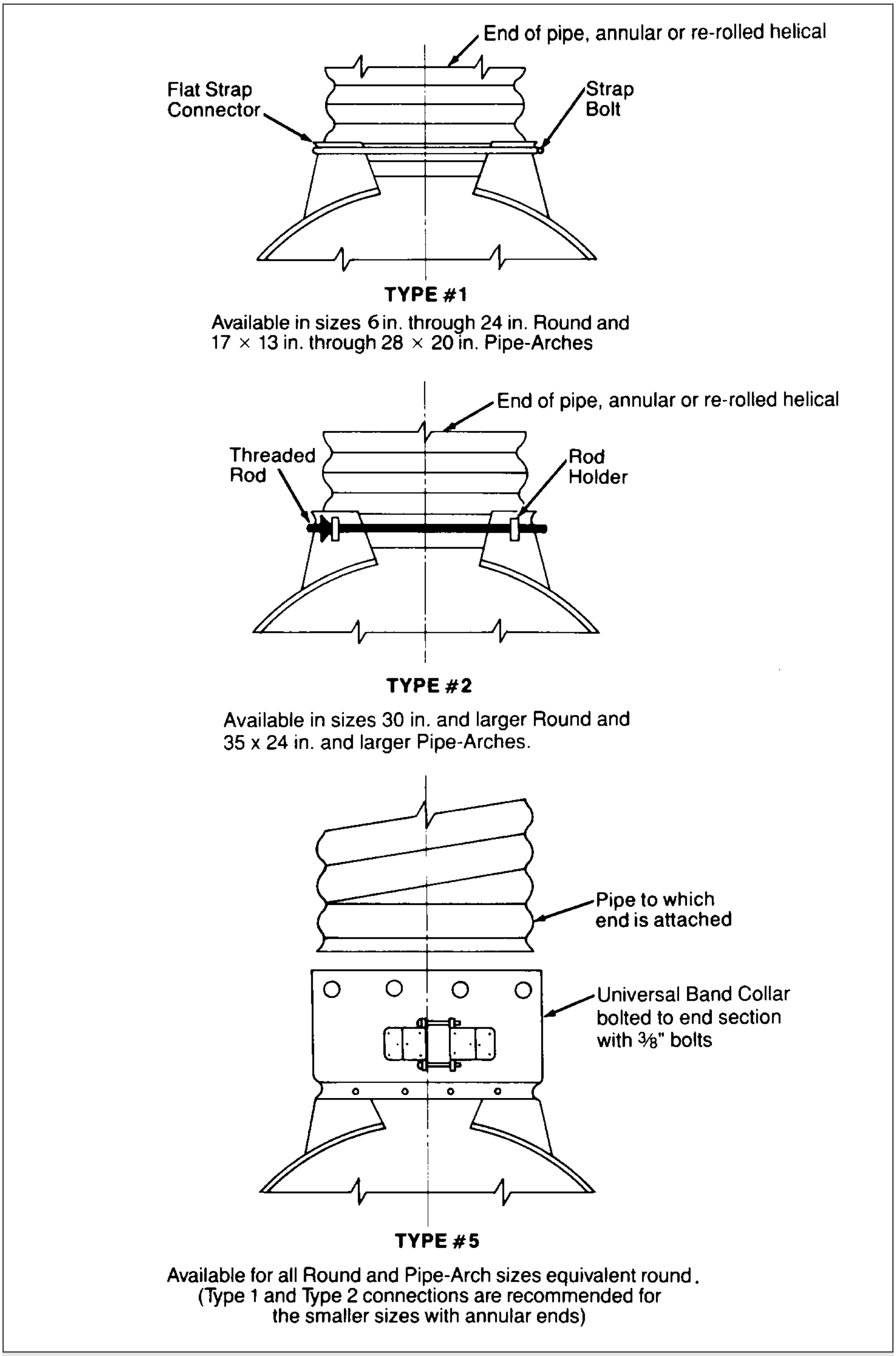
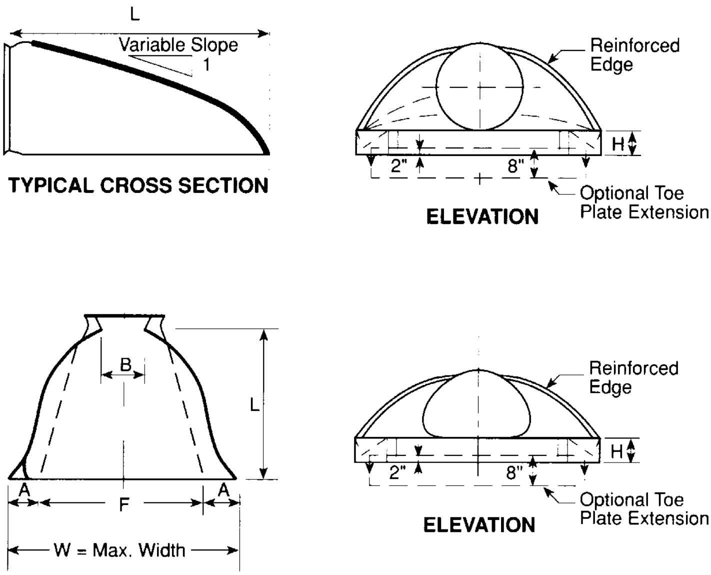


Figure 2.30 End section connection details for round and pipe arch shapes.

Table 2.57

Dimensions of steel end sections for round pipe



Pipe Diameter (in.)	Specified Thickness (in.)	A Min. (in.)	B Max. (in.)	H Min. (in.)	F Min. (in.)	L ± 2 (in.)	W Max Width (in.)	Average End Section Slope* (in.)
6	0.052	3	1	3	10	8	24	1 3/4
8	0.052	5	5	4	14	14	32	2 3/8
10	0.052	5	6	6	18	14	39	2
12	0.064	5	7	6	22	21	44	2 1/4
15	0.064	6	8	6	28	26	52	2 1/4
18	0.064	7	10	6	34	31	58	2 1/8
21	0.064	8	12	6	40	36	66	2 1/8
24	0.064	9	1 3	6	46	41	72	2 1/8
30	0.079	11	16	8	55	51	88	2 1/8
36	0.079	13	19	9	70	60	105	2
42	0.109	15	25	10	82	69	122	2 1/8
48	0.109	17	29	12	88	78	131	2
54	0.109	17	33	12	100	84	143	2
60	0.109	17	36	12	112	87	157	1 7/8
66	0.109	17	39	12	118	87	162	1 5/8
72	0.109	17	44	12	120	87	169	1 1/2
78	0.109	17	48	12	130	87	178	1 3/8
84	0.109	17	52	12	136	87	184	1 1/3
90	0.109	17	58	12	142	87	188	1 1/4
96	0.109	17	58	12	144	87	197	1 1/8

Notes: *Fill slope need not match the end section slope. Fill can be shaped at each site to fit.

1. End sections available in galvanized steel or aluminized steel, Type 2.
2. Some larger sizes may require field assembly.
3. Optional toe plates may be provided to depths specified.

Table 2.58

Dimensions of steel end sections for pipe arch 2 2/3 x 1/2 in. corrugations

Span x Rise (in.)	Equiv/ Round (in.)	Specified Thickness (in.)	A Min. (in.)	B Max. (in.)	H Min. (in.)	F Min. (in.)	L ± 2 (in.)	W Max Width (in.)	Approx. Average End Section Slope* (in.)
17 x 13	15	0.064	5	9	6	28	20	52	21/8
21 x 15	18	0.064	6	11	6	34	24	58	2
24 x 18	21	0.064	7	12	6	40	28	63	21/8
28 x 20	24	0.064	7	16	6	46	32	70	2
35 x 24	30	0.079	9	16	6	58	39	85	17/8
42 x 29	36	0.079	11	18	7	73	46	104	17/8
49 x 33	42	0.109	12	21	9	82	53	117	13/4
57 x 38	48	0.109	16	26	12	88	62	132	17/8
64 x 43	54	0.109	17	30	12	100	69	144	17/8
71 x 47	60	0.109	17	36	12	112	77	156	17/8
77 x 52	66	0.109	17	36	12	124	77	167	15/8
83 x 57	72	0.109	17	44	12	130	77	177	11/2

Notes: *Fill slope need not match the end section slope. Fill can be shaped at each site to fit.
1. End sections available in galvanized steel or aluminized steel, Type 2.
2. Some larger sizes may require field assembly.
3. Optional toe plates may be provided to depths specified.

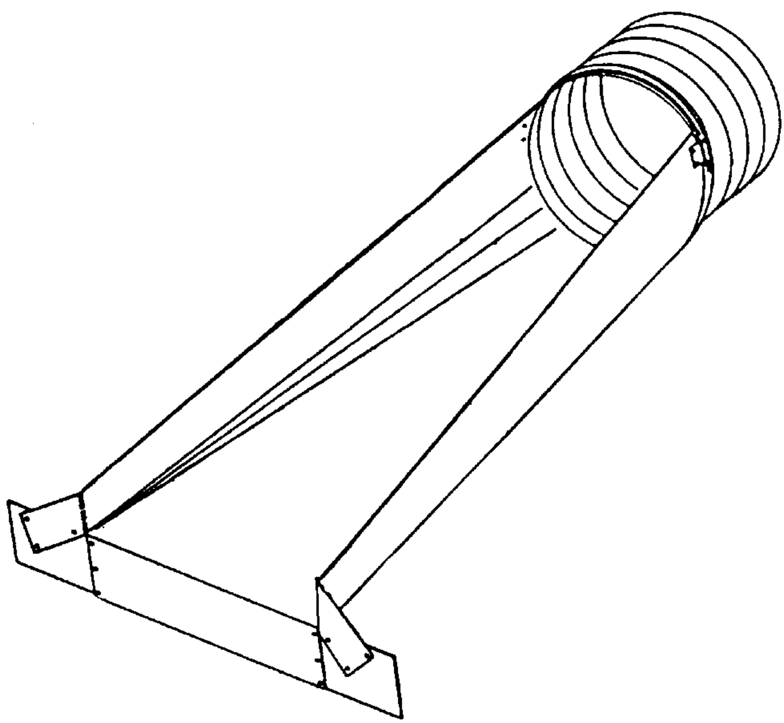
Table 2.59

Dimensions of steel end sections for pipe arch 3 x 1 in. and 5 x 1 in. corrugations

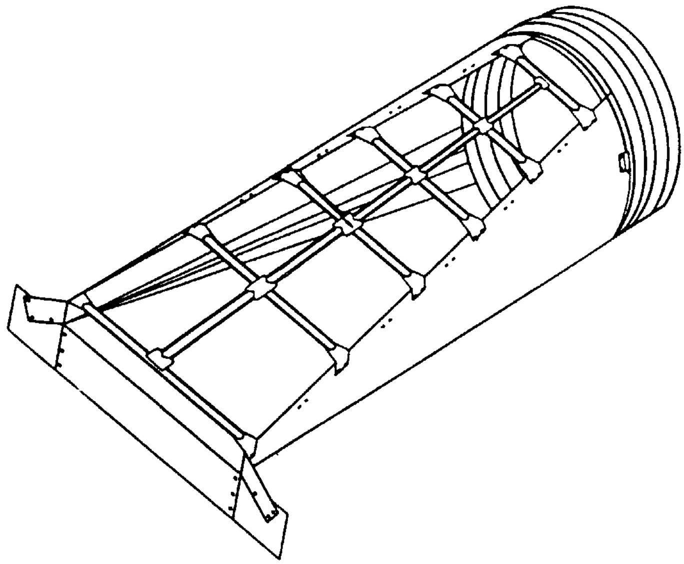
Span x Rise (in.)	Equiv/ Round (in.)	Specified Thickness (in.)	A Min. (in.)	B Max. (in.)	H Min. (in.)	F Min. (in.)	L ± 2 (in.)	W Max Width (in.)	Approx. Average End Section Slope* (in.)
53 x 41	48	0.109	17	26	12	88	63	130	13/4
60 x 46	54	0.109	17	36	12	100	70	142	17/8
66 x 51	60	0.109	17	36	12	112	77	156	13/4
73 x 55	66	0.109	17	36	12	124	77	168	11/2
81 x 59	72	0.109	17	44	12	136	77	179	15/8
87 x 63	78	0.109	17	44	12	136	77	186	11/2
95 x 67	84	0.109	1 7	44	1 2	1 60	87	210	11/2
103 x 71	90	0.109	17	44	12	172	87	222	11/3
112 x 75	96	0.109	17	44	12	172	87	226	11/4
117 x 79	102	0.109	20	62	12	154	87	234	1 1/2
128 x 83	108	0.109	20	68	12	176	87	256	1 1/2
137 x 87	114	0.109	20	73	12	194	100	274	1 1/2
142 x 91	120	0.109	20	75	12	204	98	284	1 1/2

Notes: *Fill slope need not match the end section slope. Fill can be shaped at each site to fit.
1. End sections available in galvanized steel or aluminized steel, Type 2.
2. Some larger sizes may require field assembly.
3. Optional toe plates may be provided to depths specified.

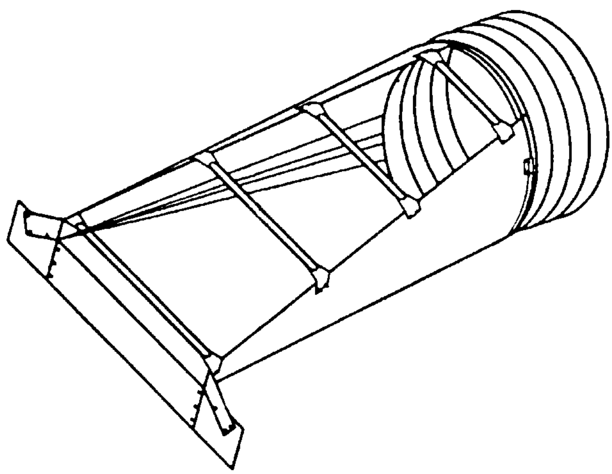
2. Safety Slope End Sections. State and federally sponsored research studies show that flatter slopes on roadside embankments greatly minimize the hazard potential to motorists. Application of this concept, with the design of 4 to 1, 6 to 1, and 10 to 1 roadside embankments, has contributed significantly to improving the safety of our highways. The use of safety slope end sections on highway culverts maintains the safety design of the flattened roadway embankments. See figures 2.31 - 2.33. The pre-fabricated safety slope end sections are available with 4 to 1, 6 to 1, and 10 to 1 slopes and are designed to fit round pipe sizes from 12 inches through 60 inches and pipe arch sizes from 17 x 13 inches through 83 inches x 57 inches While safety is the primary reason for using safety slope end sections, the tapered flare improves the hydraulic efficiency of the culvert at both the inlet and outlet ends. A deep skirt anchors the end section while preventing scour and undercutting. The flat apron or bottom panels eliminate twisting or misalignment of the end treatment. Motorists who encroach on these flattened slopes, defined as recoverable slopes, generally stop their vehicles or slow them enough to return to the roadway safely. When culverts are required on these recoverable slopes they must be made traversable or present a minimal hazard to an errant vehicle. The preferred treatment is to match the slope of the culvert with the embankment slope.



■ **Figure 2.31** Safety slope end section.



■ **Figure 2.32** Cross drainage bars on safety slope end section.



■ **Figure 2.33** Parallel bars on safety slope end section.

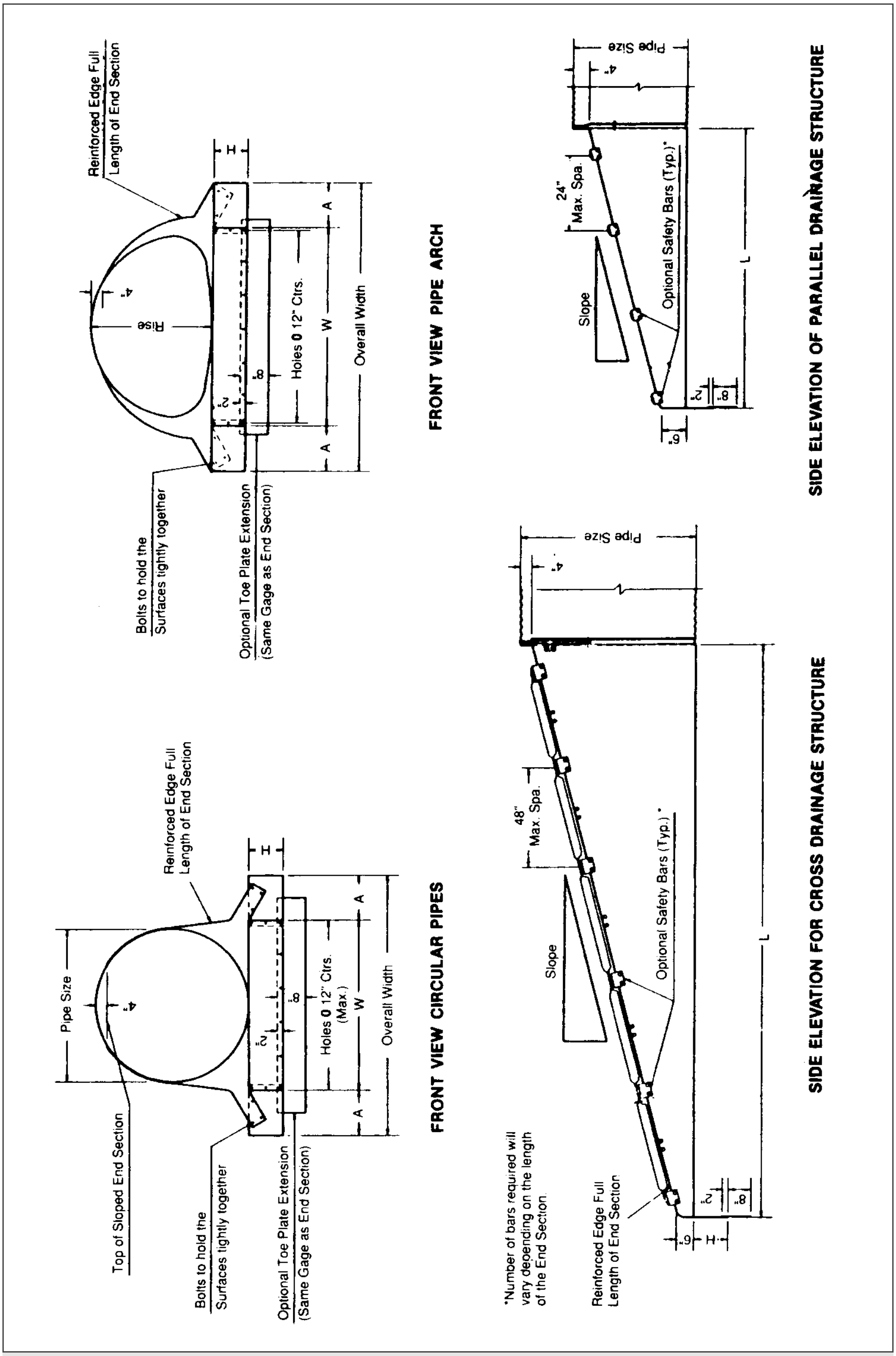
Cross drainage structures are those oriented under the main flow of traffic. On cross drainage structures, a small culvert is defined as a pipe with a 36 inch span or less or multiple pipes with a 30 inch span or less. Safety bars are not required on 30 inch spans or less when used in a cross drain application. Single structures with end sloped sections having spans greater than 36 inches can be made traversable for passenger size vehicles by using 3 inch safety bars to reduce the clear opening spans. The use of safety bars to make the safety slope end sections traversable should not decrease the hydraulic capacity of the culvert.

As referenced by AASHTO, full scale crash tests have shown that passenger size vehicles can traverse cross drainage structures with safety slope end sections equipped with cross drainage bars. This work has shown when bars are spaced on 30 inch centers, automobiles can safely cross at speeds as low as 20 mph and as high as 60 mph.

Parallel drainage structures are those oriented parallel to the main flow of traffic. They typically are used under driveways, field entrances, access ramps, intersecting side roads and median crossovers. These culverts present a significant safety hazard because they can be struck head-on by impacting vehicles. As with cross drains, the end treatments on parallel drains should match the traversable slope. Research shows that for parallel drainage structures, 3 inch diameter safety bars set on 24 inch centers will significantly reduce wheel snagging.

Safety slope end sections are efficient and provide an attractive end finish on cross and parallel drainage structures. They attach to the culvert end by simple bolted connections and can be completely salvaged if lengthening of the structure or relocation is required. Dimensions and other data are given in Tables 2.60, 2.61 and 2.62.

Round pipe with flared end sections and head wall.



■ **Figure 2.34** Safety slope end section details for round and pipe arch shapes.

Table 2.60

Dimensions of safety slope end sections for round pipe.
2 2/3 x 1/2 in., 3 x 1 in. and 5 x 1 in. corrugations

Pipe Dia. (in.)	Specified Thickness (in.)	Dimensions (in.)				L Dimensions					
		A	H	W	Overall Width	Slope	Length (in.)	Slope	Length (in.)	Slope	Length (in.)
12	.064	8	6	18	34	4:1	N/A	6:1	29		
15	.064	8	6	21	37	4:1	20	6:1	30	10:1	70
18	.064	8	6	24	40	4:1	32	6:1	48	10:1	100
21	.064	8	6	27	43	4:1	44	6:1	66	10:1	130
24	.064	8	6	30	46	4:1	56	6:1	84	10:1	160
30	.109	12	9	36	60	4:1	80	6:1	120	10:1	220
36	.109	12	9	42	66	4:1	104	6:1	156	10:1	280
42	.109	16	12	48	80	4:1	128	6:1	192		
48	.109	16	12	54	86	4:1	152	6:1	228		
54	.109	16	12	60	92	4:1	176	6:1	264		
60	.109	16	12	66	98	4:1	200	6:1	300		
66	0.109	16	12	72	104	4:1	224				
72	0.109	16	12	78	110	4:1	248				

Table 2.61

Dimensions of slope end sections for pipe arch.
2 2/3 x 1/2 in. corrugations

Pipe Dia. (in.)	Span x Rise (in.)	Specified Thickness (in.)	Dimensions (in.)				L Dimensions					
			A	H	W	Overall Width	Slope	Length (in.)	Slope	Length (in.)	Slope	Length (in.)
15	17 x 13	.064	8	6	23	39	4:1	20	6:1	30	10:1	70
18	21 x 15	.064	8	6	27	43	4:1	20	6:1	30	10:1	70
21	24 x 18	.064	8	6	30	46	4:1	32	6:1	48	10:1	100
24	28 x 20	.064	8	6	34	50	4:1	40	6:1	60	10:1	120
30	35 x 24	.079	12	9	41	65	4:1	56	6:1	84	10:1	160
36	42 x 29	.109	12	9	48	72	4:1	76	6:1	114	10:1	210
42	49 x 33	.109	16	12	55	87	4:1	92	6:1	138		
48	57 x 38	.109	16	12	63	95	4:1	112	6:1	168		
54	64 x 43	.109	16	12	70	102	4:1	132	6:1	198		
60	71 x 47	.109	16	12	77	109	4:1	148	6:1	222		
72	83 x 57	.109	16	12	89	121	4:1	188	6:1	282		

Notes:
1. End sections available in galvanized steel or aluminized steel, Type 2.
2. Cross bars and parallel bars are 3 in. Schedule 40 galvanized pipe with flattened ends bent to match end section contour.
3. Edge of side wall to be rolled edges reinforced with a 7/16 in. diameter or #4 galvanized steel rod.
4. For attachment to structure use Type 1 for circular pipe through 24 in. diameter, use Type 2 for 30 in. and larger circular pipes and all arch pipes (see Figure 2.29).

Table 2.62

Dimensions of metal slope end sections for pipe arch.
3 x 1 in. corrugations

Equiv. Dia. (in.)	Span x Rise (in.)	Specified Thickness (in.)	Dimensions (in.) A	H	W	Overall Width	L Dimensions Slope	Length (in.)	Slope	Length (in.)
48	53 x 41	0.109	16	12	59	91	4:1	132	6:1	198
54	60 x 48	0.109	16	12	66	98	4:1	152	6:1	228
60	66 x 51	0.109	16	12	72	104	4:1	172	6:1	258
66	73 x 55	0.109	16	12	79	111	4:1	188	6:1	282
72	81 x 59	0.109	16	12	87	119	4:1	204	6:1	306

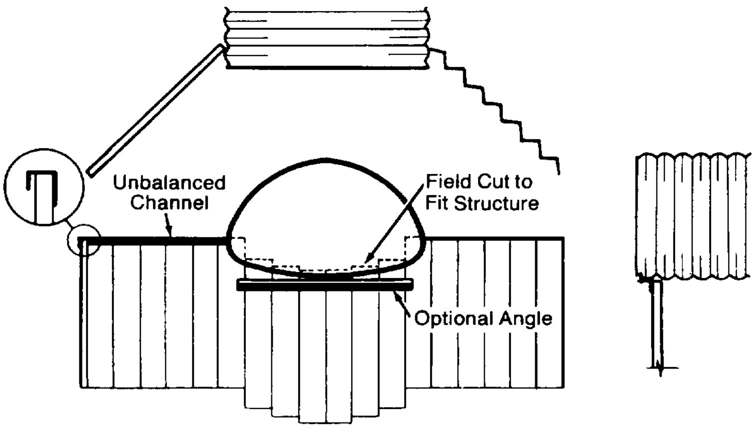
Notes: 1. End sections available in galvanized steel or aluminized steel, Type 2.
2. Cross bars and parallel bars are 3 in. Schedule 40 galvanized pipe with flattened ends bent to match end section contour.
3. Edge of side wall to be rolled edges reinforced with a 7/16 in. diameter or #4 galvanized steel rod.
4. For attachment to structure use Type 2 (see Figure 2.29).

3. Other Protection. The slope at the end of a culvert (mitered or square cut) can be protected economically against erosion by riprap, gabions and other means. Stone riprap may be sealed by portland cement grout or asphaltic concrete.

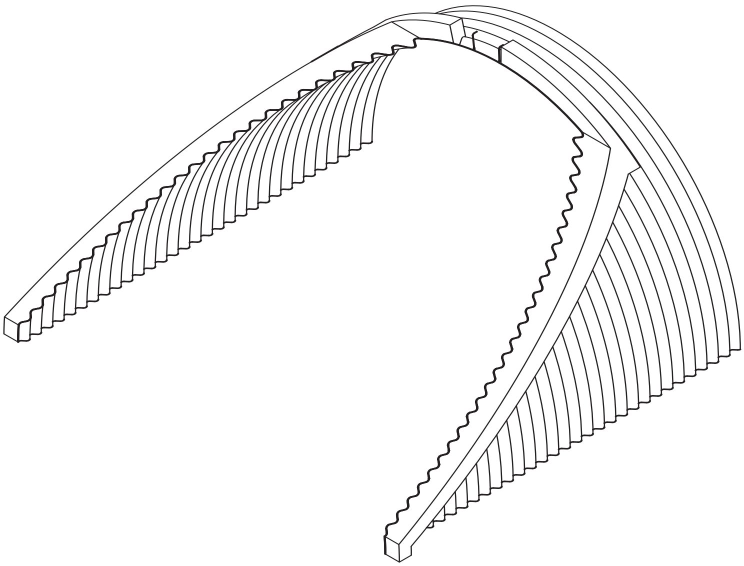
4. Skews and Bevels. Skew and bevel ends may be ordered to fit local conditions, or may consist of a standard design as shown in Figures 2.36, 2.37 and 2.38. Details and essential considerations are discussed in Chapter 8, Special Design.

5. Steel Sheeting. One practical form of end protection consists of driving corrugated steel sheeting as a cutoff wall and low height headwall or endwall. It is cut to receive the last section of the culvert barrel, and capped at about mid-diameter with an unbalanced steel channel, as shown in Figure 2.35. This type of end finish is particularly appropriate for large culverts which may have the ends beveled or step beveled. Length of the sheeting below the flow line should be one-half to one diameter of the culvert, with a minimum of 3 feet.

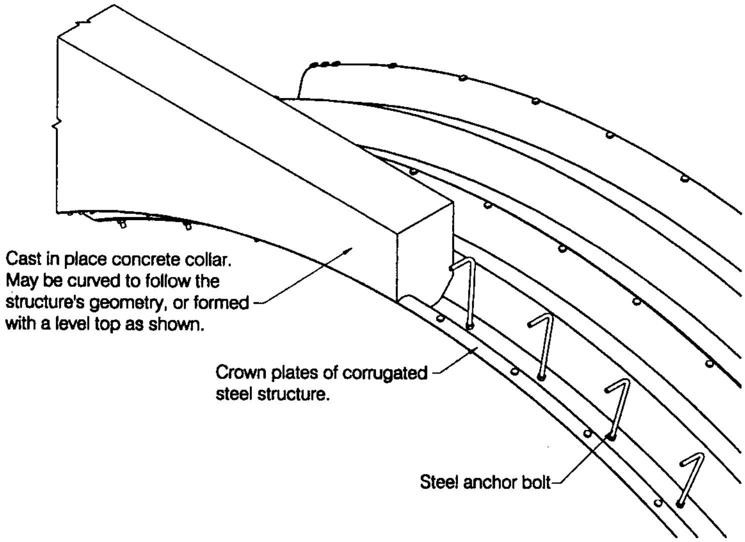
■ Standard end sections.



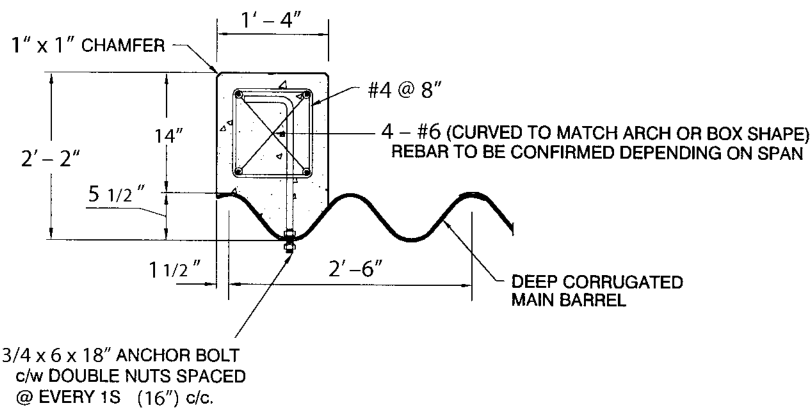
■ **Figure 2.35** How a steel sheeting headwall can be provided on a pipe arch culvert.



■ **Figure 2.36** Typical concrete collar on a structural plate arch with a beveled end.



■ **Figure 2.37** Concrete end reinforcing collar.



■ **Figure 2.38** Concrete collar detail – 15 x 5 1/2 in. structural plate.

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Deep corrugated structural plate arch during high flow.

CHAPTER three

Hydrology

INTRODUCTION

The hydrological cycle is a continuous process whereby water precipitates from the atmosphere and is transported from ocean and land surfaces back to the atmosphere from which it again precipitates. There are many inter-related phenomena involved in this process as conceptualized in Figure 3.1.

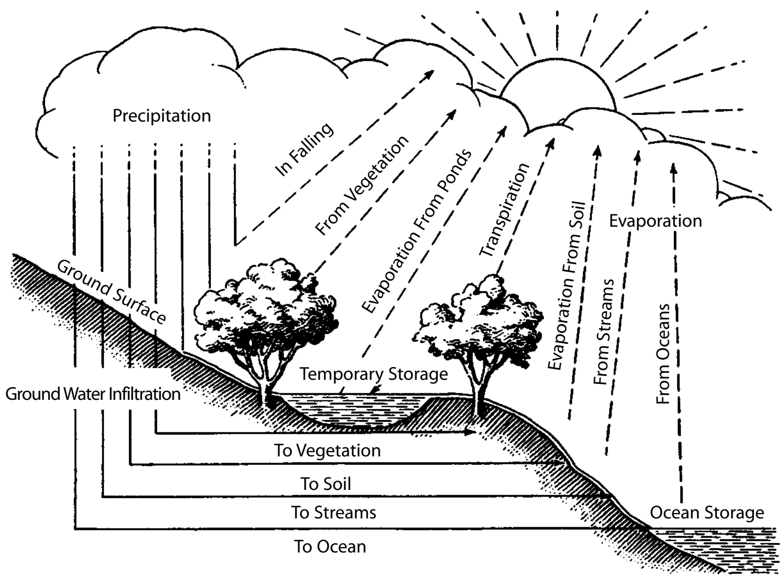
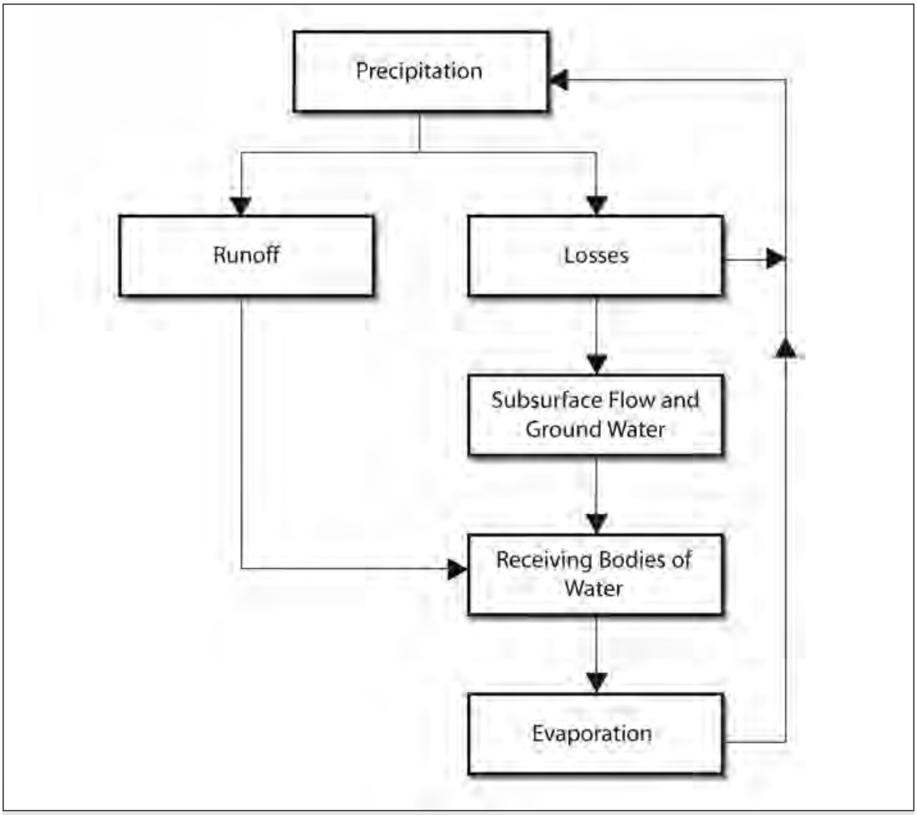


Figure 3.1 Hydrologic Cycle - where water comes from and where it goes. From M.G. Spangler's "Soil Engineering".

Different specialist interests, such as meteorologists, oceanographers or agronomists, focus on different components of the cycle. From the point of view of the drainage engineer, the relevant part of the cycle can be represented in idealistic fashion by the block diagram of Figure 3.2.

Urbanization complicates that part of the hydrologic cycle which is affected by the modifications of natural drainage paths, impounding of water, diversion of storm water and the implementation of storm water management techniques.

The objective of this chapter is to introduce the drainage engineer to the methods of estimating precipitation and runoff, those components of the hydrologic cycle that affect design decisions. Emphasis is placed on the description of alternative methods for analyzing or simulating the rainfall-runoff process, particularly where these apply to comput-



■ **Figure 3.2** Block Diagram of Hydrologic Cycle.

er models. This should help the user of these models in determining appropriate data and in interpreting the results, thereby lessening the "black box" impression that users often have.

It is often necessary to describe many of these processes in mathematical terms. Every effort has been made to keep the presentation simple but some fundamental knowledge of hydrology has been assumed.

ESTIMATION OF RAINFALL

The initial data required for drainage design is a description of the rainfall. In most cases this will be a single event storm, i.e., a period of significant and continuous rainfall preceded and followed by a reasonable length of time during which no rainfall occurs. Continuous rainfall records extending many days or weeks may sometimes be used for the simulation of a series of storms, particularly where the quantity rather than the quality of runoff water is of concern.

The rainfall event may be either historic, taken from recorded events, or idealized. The main parameters of interest are the total amount (or depth) of precipitation (P_{tot}), the duration of the storm (t_d), and the distribution of the rainfall intensity (i) throughout the storm event. The frequency of occurrence (N) of a storm is usually expressed in years. It is an estimate based on statistical records of the long-term average time interval expected to elapse between successive occurrences of two storms of a particular severity. For example, a storm of depth P_{tot} with a duration of t_d is expected to occur, on average, every N years. The word "expected" is emphasized because there is absolutely no certainty that after a 25-year storm has occurred, a storm of equal or greater severity will not occur for another 25 years. This fact, while statistically true, is often difficult to convey to concerned or affected citizens.

Rainfall Intensity-Duration-Frequency Curves

Rainfall intensity-duration-frequency (IDF) curves are derived from the statistical analysis of rainfall records compiled over a number of years. Each curve represents the intensity-time relationship for a certain return frequency, from a series of storms. These curves are then said to represent storms of a specific return frequency.

The intensity, or the rate of rainfall, is usually expressed in depth per unit time. The frequency of occurrence (N), in years, is a function of the storm intensity. Larger storm intensities occur less frequently. The highest intensity for a specific duration of N years of records is called the N year storm, with a frequency of once in N years.

The curves may be in graphical form as shown in Figure 3.3, or may be represented by individual equations that express the time-intensity relationships for specific frequencies. Such equations are in the form

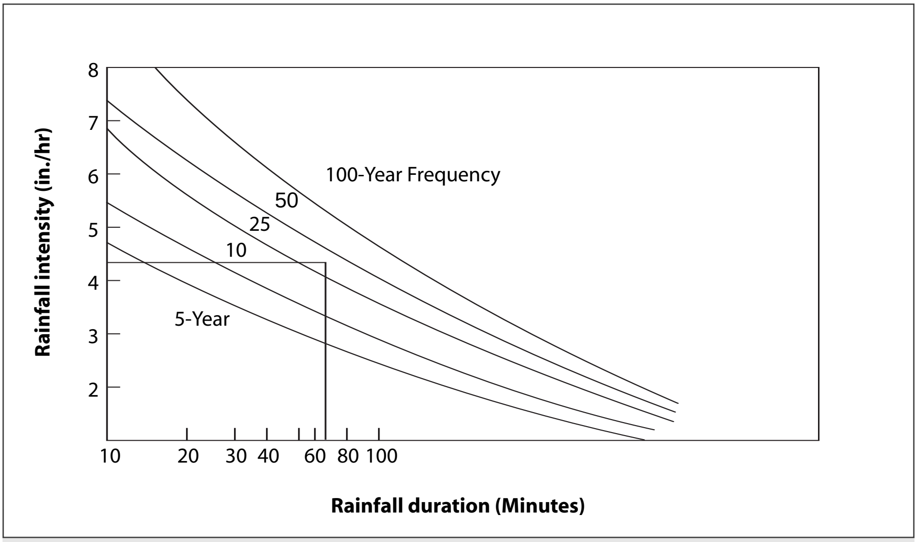
$$i = \frac{a}{(t + c)^b}$$

where: i = intensity (in./hr)
t = time (minutes)
a,b,c = constants developed for each IDF curve

The fitting of rainfall data to the equation may be obtained by either graphical or least square methods.

It should be noted that the IDF curves do not represent a rainfall pattern, but are the distribution of the highest intensities over time durations for a storm of N frequency.

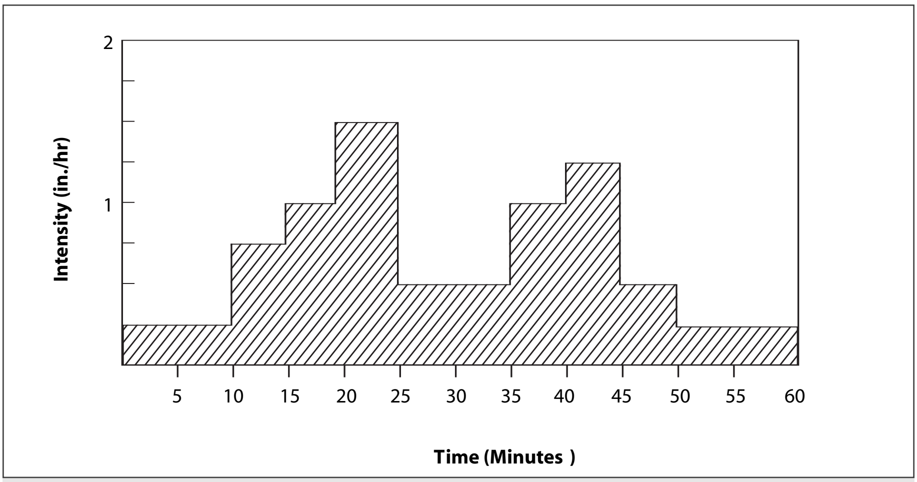
The rainfall intensity-duration-frequency curves are readily available from governmental agencies, local municipalities and towns, and are therefore widely used for designing drainage facilities and flood flow analysis.



■ **Figure 3.3** Rainfall intensties for various storm frequencies vs. rainfall duration.

Rainfall Hyetographs

The previous section discussed the dependence of the average rainfall intensity of a storm on various factors. It is also important to consider, from historical rainfall events, the way in which the precipitation is distributed in time over the duration of the storm. This can be described using a rainfall hyetograph which is a graphical representation of the variation of rainfall intensity with time. Rainfall hyetographs can be obtained (usually in tabular rather than graphical form) from weather stations that have suitable records of historical rainfall events. Figure 3.4 shows a typical example.



■ **Figure 3.4** Rainfall hyetograph.

Rainfall intensity is usually plotted in the form of a bar graph. It is therefore assumed that the rainfall intensity remains constant over the timestep (Figure 3.4) used to describe the hyetograph. This approximation becomes a truer representation of reality as the timestep gets smaller. However, very small timesteps may require very large amounts of data to represent a storm. At the other extreme, it is essential that the timestep not be too large, especially for short duration events or for very small catchments. Otherwise, peak values of both rainfall and runoff can be "smeared" with consequent loss of accuracy. This point should be kept in mind, when using a computer model, since it is standard practice to employ the same timestep for the description of the rainfall hyetograph and for the computation of the runoff hyetograph. Choice of a timestep is therefore influenced by:

a) accuracy of rainfall-runoff representation,
b) the number of available data points, and
c) size of the watershed.

Synthetic Rainfall Hyetographs

An artificial or idealized hyetograph may be required for a number of reasons, two of which are:

a) The historic rainfall data may not be available for the location or the return frequency desired.

b) It may be desirable to standardize the design storm to be used within a region so that comparisons of results from various studies may be made.

The time distribution of the selected design hyetograph will significantly affect the timing and magnitude of the peak runoff. Therefore, care should be taken in selecting a design storm to ensure that it is representative of the rainfall patterns in the area under study. In many cases, depending upon the size of the watershed and degree of urbanization, it may be necessary to use several different rainfall hyetographs to determine the sensitivity of the results to the different design storms. For example, when runoff from pervious areas is significant, it will be found that late peaking storms produce a higher peak runoff than early peaking storms of the same total depth. Early peaking storms are reduced in severity by the initially high infiltration capacity of the ground.

Selection of the storm duration will also influence the hyetograph characteristics. The handbook of the Natural Resource Conservation Service (formerly Soil Conservation Service) recommends that a six hour storm duration be used for watersheds with a time of concentration (which is discussed in detail later in this chapter) less than or equal to six hours. For watersheds where the time of concentration exceeds six hours, the storm duration should equal the time of concentration.

A number of different synthetic hyetographs are described in the following sections. These include:

a) uniform rainfall (as in the Rational Method),
b) the Chicago hyetograph,
c) Huff's rainfall distribution curves, and
d) SCS design storms.

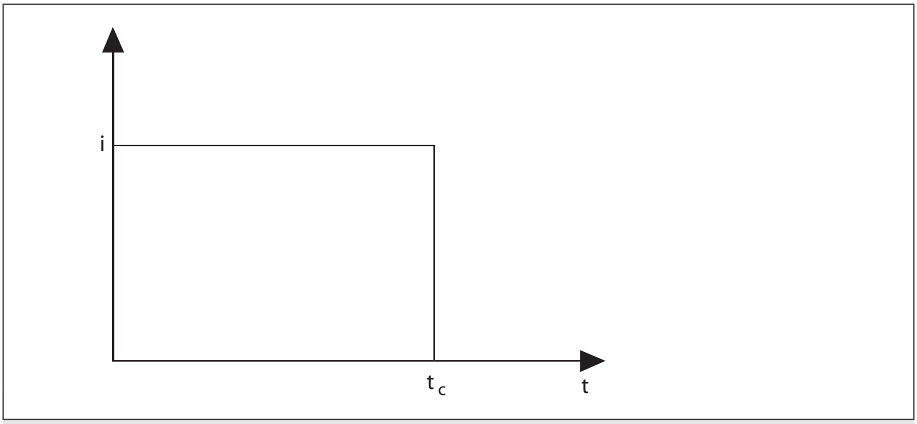
Uniform Rainfall

The simplest possible design storm is to assume that the intensity is uniformly distributed throughout the storm duration. The intensity is then represented by

$$i = i_{ave} = \frac{P_{tot}}{t_d}$$

where: P_{tot} = total precipitation
t_d = storm duration

This simplified approximation is used in the Rational Method, assuming that the storm duration, td, is equal to the time of concentration, tc, of the catchment (see Figure 3.5). A rectangular rainfall distribution is only used for approximations or rough estimates. It can, however, have some use in explaining or visualizing rainfall-runoff processes, since any hyetograph may be considered as a series of such uniform, short duration pulses of rainfall.



■ **Figure 3.5** Uniform rainfall intensity.

Chicago Hyetograph

The Chicago hyetograph is assumed to have a time distribution such that if a series of ever increasing "time-slices" were analyzed around the peak rainfall, the average intensity for each "slice" would lie on a single IDF curve. Therefore, the Chicago design storm displays statistical properties which are consistent with the statistics of the IDF curve. That being the case, the synthesis of the Chicago hyetograph starts with the parameters of an IDF curve together with a parameter (r) which defines the fraction of the storm duration that occurs before the peak rainfall intensity. The value of r is derived from the analysis of actual rainfall events and is generally in the range of 0.3 to 0.5.

The continuous curves of the hyetograph in Figure 3.6 can be computed in terms of the times before (t_b) and after (t_a) the peak intensity by the two equations below.

After the peak:

$$i_a = \frac{a\left[(1-b)\dfrac{t_a}{1-r} + c\right]}{\left(\dfrac{t_a}{1-r} + c\right)^{1+b}}$$

Before the peak:

$$i_b = \frac{a\left[(1-b)\dfrac{t_b}{r} + c\right]}{\left(\dfrac{t_b}{r} + c\right)^{1+b}}$$

where: t_a = time after peak
t_b = time before peak
r = ratio of time before the peak occurs to the total duration time (the value is derived from analysis of actual rainfall events)

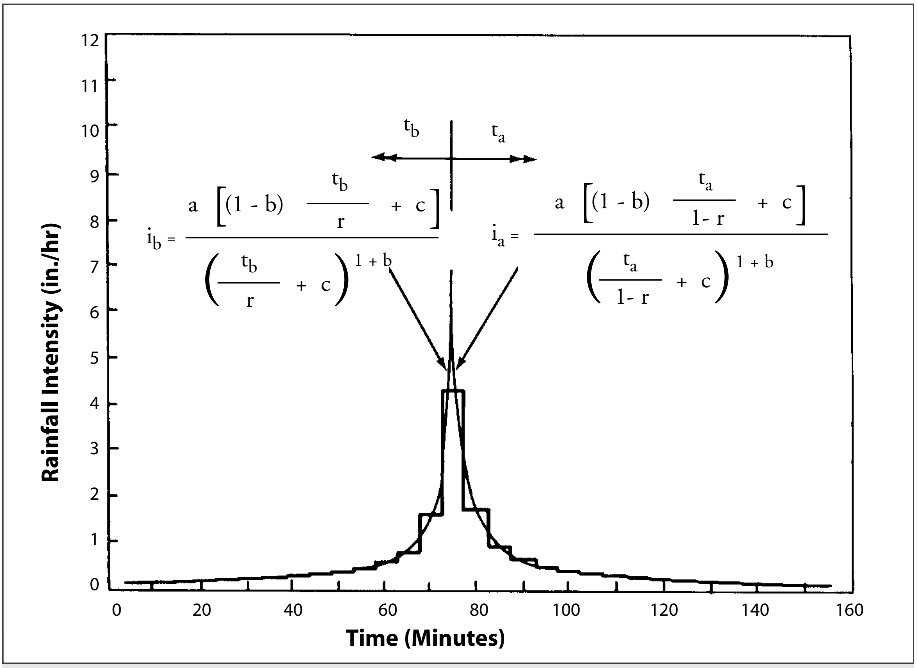


Figure 3.6 Chicago hyetograph.

The Chicago storm is commonly used for small to medium watersheds (0.1 to 10 square miles) for both rural and urban conditions. Typical storm durations are in the range of 1.0 to 4.0 hours. It has been found that peak runoff flows computed using a Chicago design storm are higher than those obtained using other synthetic or historic storms. This is due to the fact that the Chicago storm attempts to model the statistics of a large collection of real storms and thus tends to present an unrealistically extreme distribution. Also, the resultant peak runoff may exhibit some sensitivity to the timestep used; very small timesteps give rise to more peaked runoff hydrographs (which are discussed later in this chapter).

The Huff Rainfall Distribution Curves

Huff analyzed the significant storms in 11 years of rainfall data recorded by the State of Illinois. The data were represented in non-dimensional form by expressing the accumulated depth of precipitation, P_t, (that is, the accumulated depth at time t after the start of rainfall) as a fraction of the total storm depth, P_{tot}, and plotting this ratio as a function of a non-dimensional time, t/t_d, where t_d is time of duration.

The storms were grouped into four categories depending on whether the peak rainfall intensity fell in the 1^{st}, 2^{nd}, 3^{rd} or 4^{th} quartile of the storm duration. In each category, a family of curves was developed representing values exceeded in 90%, 80%, 70%, etc., of the storm events. Thus the average of all the storm events in a particular category is represented by the 50% curve. Table 3.1 shows the dimensionless coefficients for each quartile expressed at intervals of 5% of t_d.

Table 3.1

Dimensionless Huff storm coefficients

t / t_d	P_t / P_{tot} for Quartile			
	1	2	3	4
0.00	0.000	0.000	0.000	0.000
0.05	0.063	0.015	0.020	0.020
0.10	0.178	0.031	0.040	0.040
0.15	0.333	0.070	0.072	0.055
0.20	0.500	0.125	0.100	0.070
0.25	0.620	0.208	0.122	0.085
0.30	0.705	0.305	0.140	0.100
0.35	0.760	0.420	0.155	0.115
0.40	0.798	0.525	0.180	0.135
0.45	0.830	0.630	0.215	0.155
0.50	0.855	0.725	0.280	0.185
0.55	0.880	0.805	0.395	0.215
0.60	0.898	0.860	0.535	0.245
0.65	0.915	0.900	0.690	0.290
0.70	0.930	0.930	0.790	0.350
0.75	0.944	0.948	0.875	0.435
0.80	0.958	0.962	0.935	0.545
0.85	0.971	0.974	0.965	0.740
0.90	0.983	0.985	0.985	0.920
0.95	0.994	0.993	0.995	0.975
1.00	1.000	1.000	1.000	1.000

The first quartile curve is generally associated with relatively short duration storms in which 62% of the precipitation depth occurs in the first quarter of the storm duration. The fourth quartile curve is normally used for longer duration storms in which the rainfall is more evenly distributed over the duration t_d and is often dominated by a series of rain showers or steady rain or a combination of both. The third quartile has been found to be suitable for storms on the Pacific seaboard.

The study area and storm duration for which the distributions were developed vary considerably, with t_d varying from 3 to 48 hours and the drainage basin area ranging from 10 to 400 square miles. The distributions are most applicable to Midwestern regions of North America and regions of similar rainfall climatology and physiography.

To use the Huff distribution the user need only specify the total depth of rainfall (P_{tot}), the duration (t_d) and the desired quartile. The curve can then be scaled up to a dimensional mass curve and the intensities are obtained from the mass curve for the specified timestep (t).

SCS Design Storms

The U.S. Soil Conservation Service (SCS) design storm was developed for various storm types, storm durations and regions of the United States. The storm duration was initially selected to be 6 hours. Durations of 3 hours and up to 48 hours have, however, been

Table 3.2

SCS Type II rainfall distribution for 3h, 6h, 12h and 24h durations

3 Hour			6 Hour			12 Hour			24 Hour		
Time ending	F_{inc} (%)	F_{cum} (%)	Time ending	F_{inc} (%)	F_{cum} (%)	Time ending	F_{inc} (%)	F_{cum} (%)	Time ending	F_{inc} (%)	F_{cum} (%)
						0.5	1	1			
			0.5	2	2	1.0	1	2	2	2	2
						1.5	1	3			
0.5	4	4	1.0	2	4	2.0	1	4	4	2	4
						2.5	2	6			
			1.5	4	8	3.0	2	8	6	4	8
						3.5	2	10			
1.0	8	12	2.0	4	12	4.0	2	12	8	4	12
						4.5	3	15			
			2.5	7	19	5.0	4	19	10	7	19
						5.5	6	25			
1.5	58	70	3.0	51	70	6.0	45	70	12	51	70
						6.5	9	79			
			3.5	13	83	7.0	4	83	14	13	83
						7.5	3	86			
2.0	19	89	4.0	6	89	8.0	3	89	16	6	89
						8.5	2	91			
			4.5	4	93	9.0	2	93	18	4	93
						9.5	2	95			
2.5	7	96	5.0	3	96	10.0	1	96	20	3	96
						10.5	1	97			
			5.5	2	98	11.0	1	98	22	2	98
						11.5	1	99			
3.0	4	100	6.0	2	100	12.0	1	100	24	2	100

developed. The rainfall distribution varies depending on duration and location. The 3, 6, 12 and 24 hour distributions for the SCS Type II storm are given in Table 3.2. These distributions are used in all regions of the United States with the exception of the Pacific coast.

The design storms were initially developed for large (10 square mile) rural basins. However, the longer duration (6 to 48 hour) distributions and a shorter 1 hour duration thunderstorm distribution have been used in urban and smaller rural areas.

The longer duration storms tend to be used for sizing detention facilities while at the same time providing a reasonable peak flow for sizing the conveyance system.

ESTIMATION OF EFFECTIVE RAINFALL

Only a fraction of the precipitation that falls during a storm contributes to the overland flow or runoff from the catchment. The balance is diverted in various ways.

Evaporation In certain climates, some fraction of the rainfall evaporates before reaching the ground. Since rainfall is measured by gages on the earth's surface, this subtraction is automatically taken into account in recorded storms and may be ignored by the drainage engineer.

Interception This fraction is trapped in vegetation or roof depressions and never reaches the catchment surface. It eventually dissipates by evaporation.

Infiltration Rainfall that reaches a pervious area of the ground surface will initially be used to satisfy the capacity for infiltration into the upper layer of the soil. After even quite a short dry period the infiltration capacity can be quite large (for example, 4 in./hr) but this gradually diminishes after the start of rainfall as the storage capacity of the ground is saturated. The infiltrated water will:

a) evaporate directly by capillary rise,
b) rise through the root system and be transpired from vegetal cover where it then evaporates,
c) move laterally through the soil in the form of ground water flow toward a lake or a stream, and/or
d) penetrate to deeper levels to recharge the ground water.

Surface Depression Storage If the intensity of the rainfall reaching the ground exceeds the infiltration capacity of the ground, the excess will begin to fill the small depressions on the ground surface. Clearly this will begin to happen almost immediately on impervious surfaces. Only after these tiny reservoirs have been filled will overland flow commence and contribute to the runoff from the catchment. Since these surface depressions are not uniformly distributed, it is quite possible that runoff will commence from some fraction of the catchment area before the depression storage on another fraction is completely filled. Typical recommended values for surface depression storage are given in Table 3.3.

Table 3.3

Typical recommended values for depth of surface depression storage

Land Cover	Recommended Value (in.)
Large Paved Areas	0.1
Roofs, Flat	0.1
Fallow Land Field without Crops	0.2
Fields with Crops (grain, root crops)	0.3
Grass Areas in Parks, Lawns	0.3
Wooded Areas and Open Fields	0.4

The effective rainfall is thus that portion of the storm rainfall that contributes directly to the surface runoff hydrograph. This can be expressed as follows:

Runoff = Precipitation - Interception - Infiltration - Surface Depression Storage

All of the terms are expressed in units of depth.

A number of methods are available to estimate the effective rainfall and thus the amount of runoff for any particular storm event. These range from the runoff coefficient (C) of the Rational Method to relatively sophisticated computer implementations of semi-empirical methods representing the physical processes. The method selected should be based on the size of the drainage area, the data available and the degree of sophistication warranted for the design. Three methods for estimating effective rainfall are:

1) the Rational Method,
2) the Soil Conservation Service (SCS) Method, and
3) the Horton Method.

The Rational Method

If an impervious area (A) is subjected to continuous and long lasting rainfall of a specific intensity (i), then after a time (time of concentration, T_c) the runoff rate will be given by:

$$Q = k \bullet C \bullet i \bullet A$$

where:
Q = peak runoff rate (ft^3/s)
k = constant = 1.0 for U.S. Customary Units
C = runoff coefficient
i = rainfall intensity (in./hr)
A = drainage area (acres)

When using the Rational Method, the following assumptions are considered:

a) The rainfall intensity is uniform over the entire watershed during the entire storm duration.
b) The maximum runoff rate occurs when the rainfall lasts as long or longer than the time of concentration.
c) The time of concentration is the time required for the runoff from the most remote part of the watershed to reach the point under design.

The variable C is the component of the Rational Method that requires the most judgment, and the runoff is directly proportional to the value assigned to C. Care should be exercised in selecting the value as it incorporates all of the hydrologic abstractions, soil types and antecedent conditions. Table 3.4 lists typical values for C, as a function of land use, for storms that have (approximately) a 5 to 10 year return period. It is important to note that the appropriate value of C depends on the magnitude of the storm. Significantly higher values of C may be necessary for more extreme storm events. This is perhaps one of the most serious deficiencies associated with this method.

Table 3.4

Recommended runoff coefficients based on description of area

Description of Area	Runoff Coefficients
Business	
Downtown	0.70 to 0.95
Neighborhood	0.50 to 0.70
Residential	
Single-family	0.30 to 0.50
Multi-units, detached	0.40 to 0.60
Multi-units, attached	0.60 to 0.75
Residential (suburban)	0.25 to 0.40
Apartment	0.50 to 0.70
Industrial	
Light	0.50 to 0.80
Heavy	0.60 to 0.90
Parks, cemeteries	0.10 to 0.25
Playgrounds	0.20 to 0.35
Railroad yard	0.20 to 0.35
Unimproved	0.10 to 0.30

It often is desirable to develop a composite runoff coefficient based on the percentage of different types of surfaces in the drainage area. This procedure is often applied to typical "sample" blocks as a guide to the selection of reasonable values of the coefficient for an entire area. Coefficients, with respect to surface type, are shown in Table 3.5.

Table 3.5

Recommended runoff coefficients based on character of surface

Character of Surface	Runoff Coefficients
Pavement	
Asphalt and Concrete	0.70 to 0.95
Brick	0.70 to 0.85
Roofs	0.75 to 0.95
Lawns, sandy soil	
Flat, 2 percent	0.13 to 0.17
Average, 2 to 7 percent	0.18 to 0.22
Steep, 7 percent	0.25 to 0.35

The coefficients in these two tables are applicable for storms of 5 to 10 year frequencies. Less frequent, higher intensity storms will require the use of higher coefficients because infiltration and other losses have a proportionally smaller effect on runoff. The coefficients are based on the assumption that the design storm does not occur when the ground surface is frozen.

The SCS Method

Referred to here as the SCS Method, the Natural Resource Conservation Service (formerly Soil Conservation Service) developed a relationship between rainfall (P), retention (S), and effective rainfall or runoff (Q). The retention, or potential storage in the soil, is established by selecting a curve number (CN). The curve number is a function of soil type, ground cover and Antecedent Moisture Condition (AMC).

The hydrological soil groups, as defined by SCS soil scientists, are:

A. (Low runoff potential) Soils having a high infiltration rate, even when thoroughly wetted, consisting chiefly of deep, well to excessively well drained sands or gravel.

B. Soils having a moderate infiltration rate when thoroughly wetted, consisting chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse texture.

C. Soils having a slow infiltration rate when thoroughly wetted, consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine texture.

D. (High runoff potential) Soils having a very slow infiltration rate when thoroughly wetted, consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a clay pan or clay layer at or near the surface, and shallow soils over nearly impervious material.

Knowing the hydrological soil group and the corresponding land use, the runoff potential or CN value of a site may be determined. Table 3.6 lists typical CN values. Three levels of Antecedent Moisture Condition are considered in the SCS Method. The Antecedent Moisture Condition (AMC) is defined as the amount of rainfall in a period of five to thirty days preceding the design storm. In general, the heavier the antecedent rainfall, the greater the runoff potential. AMC definitions are as follows:

AMC I - Soils are dry but not to the wilting point. This is the lowest runoff potential.
AMC II - This is the average case, where the soil moisture condition is considered to be average.
AMC III - Heavy or light rainfall and low temperatures having occurred during the previous five days. This is the highest runoff potential.

Table 3.6

Runoff curve numbers for selected agricultural, suburban and urban land use (Antecedend Moisture Condition II and $I_a = 0.2$ S)

			HYDROLOGIC SOIL GROUP			
LAND USE DESCRIPTION			**A**	**B**	**C**	**D**
Cultivated land[1]:	without conservation treatment		72	81	88	91
	with conservation treatment		62	71	78	81
Pasture or range land:	poor condition		68	79	86	89
	good condition		39	61	74	80
Meadow:	good condition		30	58	71	78
Wood or forest land:	thin stand, poor cover, no mulch		45	66	77	83
	good cover[2]		25	55	70	77
Open spaces, lawns, parks, golf courses, cemeteries, etc.						
good condition:	grass cover on 75% or more of the area		39	61	74	80
fair condition:	grass cover on 50% to 75% of the area		49	69	79	84
Commercial and business areas (85% impervious)			89	92	94	95
Industrial districts (72% impervious)			81	88	91	93
Residential[3]:	Average lot size	Average % Impervious[4]				
	1/8 acre	65	77	85	90	92
	1/4 acre	38	61	75	83	87
	1/3 acre	30	57	72	81	86
	1/2 acre	25	54	70	80	85
	1 acre	20	51	68	79	84
Paved parking lots, roofs, driveways, etc.[5]			98	98	98	98
Streets and roads:	paved with curbs and storm sewers5		98	98	98	98
	gravel		76	85	89	91
	dirt		72	82	87	89

1. For a more detailed description of agricultural land use curve numbers refer to National Engineering Handbook, Section 4, Hydrology, Chapter 9, Aug 1972.
2. Good cover is protected from grazing and litter and brush cover soil.
3. Curve numbers are computed assuming the runoff from the house and driveway is directed towards the street with a minimum of roof water directed to lawns where additional infiltration could occur.
4. The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers.
5. In some warmer climates of the country, a curve number of 95 may be used.

The CN values in Table 3.6 are based on Antecedent Moisture Condition II. Thus, if moisture conditions I or III are chosen, then a corresponding CN value is determined as provided in Table 3.7.

Table 3.7

Curve number relationships for different Antecedent Moisture Conditions

CN for Condition II	CN for Condition I	CN for Condition III	CN for Condition II	CN for Condition I	CN for Condition III
100	100	100	60	40	78
99	97	100	59	39	77
98	94	99	58	38	76
97	91	99	57	37	75
96	89	99	56	56	75
95	87	98	55	35	74
94	85	98	54	34	73
93	83	98	53	33	72
92	81	97	52	32	71
91	80	97	51	31	70
90	78	96	50	31	70
89	76	96	49	30	69
88	75	95	48	29	68
87	73	95	47	28	67
86	72	94	46	27	66
85	70	94	45	26	65
84	68	93	44	25	64
83	67	93	43	25	63
82	66	92	42	24	62
81	64	92	41	23	61
80	63	91	40	22	60
79	62	91	39	21	59
78	60	90	38	21	58
77	59	89	37	20	57
76	58	89	36	19	56
75	57	88	35	18	55
74	55	88	34	18	54
73	54	87	33	17	53
72	53	86	32	16	52
71	52	86	31	16	51
70	51	85	30	15	50
69	50	84			
68	48	84	25	12	43
67	47	83	20	9	37
66	46	82	15	6	30
65	45	82	10	4	22
64	44	81	5	2	13
63	43	80	0	0	0
62	42	79			
61	41	78			

The potential storage in the soils is based on an initial abstraction (I_a) which is the interception, infiltration and depression storage prior to runoff, and infiltration after runoff.

The effective rainfall is defined by the relationship:

$$Q = \frac{(P - I_a)^2}{P + S - I_a}$$

where $S = [(100/CN) - 10] \bullet 25.4$

The original SCS Method assumed the value of I_a to be equal to 0.2S. However, many engineers have found that this may be overly conservative, especially for moderate rainfall events and low CN values. Under these conditions, the I_a value may be reduced to be a lesser percentage of S or may be estimated and input directly into the above equation.

The Horton Method

The Horton infiltration equation, which defines the infiltration capacity of the soil, changes the initial rate (f_o) to a lower rate (f_c). The "maximum infiltration capacity" infiltration rate, calculated using the equation, only occurs when the available rainfall equals or exceeds that infiltration rate. Until that occurs, the infiltration rate is the rainfall intensity. Therefore, if the infiltration capacity is given by:

$$f_{cap} = f_c + (f_o - f_c)\, e^{-t \bullet k}$$

then the actual infiltration (f), will be defined by one of the following two equations:

$$f = f_{cap} \quad \text{for } i \geq f_{cap}$$

$$f = i \quad \text{for } i \leq f_{cap}$$

where:
- f = actual infiltration rate into the soil
- f_{cap} = maximum infiltration capacity of the soil
- f_o = initial infiltration capacity
- f_c = final infiltration capacity
- i = rainfall intensity
- k = exponential decay constant (1/hours)
- t = elapsed time from start of rainfall (hours)

Figure 3.7 shows a typical rainfall distribution and infiltration curve.

For the initial timesteps, the infiltration rate exceeds the rainfall rate. The reduction in infiltration capacity is dependent more on the reduction in storage capacity in the soil rather than the elapsed time from the start of rainfall. To account for this, the infiltration curve should, therefore, be shifted (dashed line for first timestep, Δt) by an elapsed time that would equate the infiltration volume to the volume of runoff.

A further modification is necessary if surface depression is to be accounted for. Since the storage depth must be satisfied before overland flow can occur, the initial finite values of the effective rainfall hyetograph must be reduced to zero until a depth equivalent to the surface depression storage has been accumulated. The final hyetograph is the true effective rainfall that will generate runoff from the catchment surface.

The selection of the parameters for the Horton equation depends on soil type, vegetal cover and Antecedent Moisture Conditions. Table 3.8 shows typical values for f_o and f_c

(in./hour) for a variety of soil types under different crop conditions. The value of the lag constant should typically be between 0.04 and 0.08.

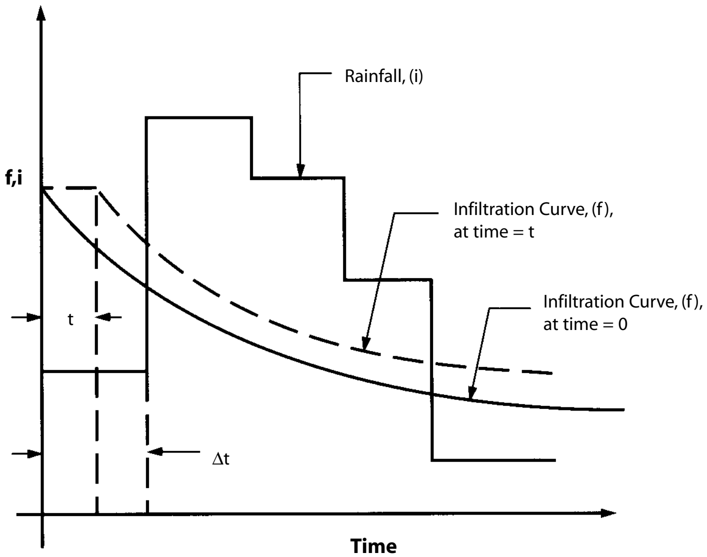


Figure 3.7 Representation of the Horton equation.

Table 3.8

Typical values for the Horton equation parameters (in./hr)

	Loam, Clay K = 0.08		Clayey Sand K = 0.06		Sand, Loess, Gravel K = 0.04	
Land Surface Types	f_o	f_c	f_o	f_c	f_o	f_c
Fallow land field without crops	15	8	33	10	43	15
Fields with crops (grain, root crops, vines)	36	3	43	8	64	10
Grassed verges, playground, ski slopes	20	3	20	3	20	3
Noncompacted grassy surface, grass areas in parks, lawns	43	8	64	10	89	18
Gardens, meadows, pastures	64	10	71	15	89	18
Coniferous woods	53*	53*	71*	71*	89*	89*
City parks, woodland, orchards	89	53	89	71	89*	89*

Notes: *K=0

Comparison of the SCS and Horton Methods

Figure 3.8 illustrates the various components of the rainfall-runoff process for the SCS and Horton Methods. The following example serves to demonstrate the difference between the SCS Method, in which the initial abstraction is used, and the moving curve Horton Method, in which surface depression storage is significant.

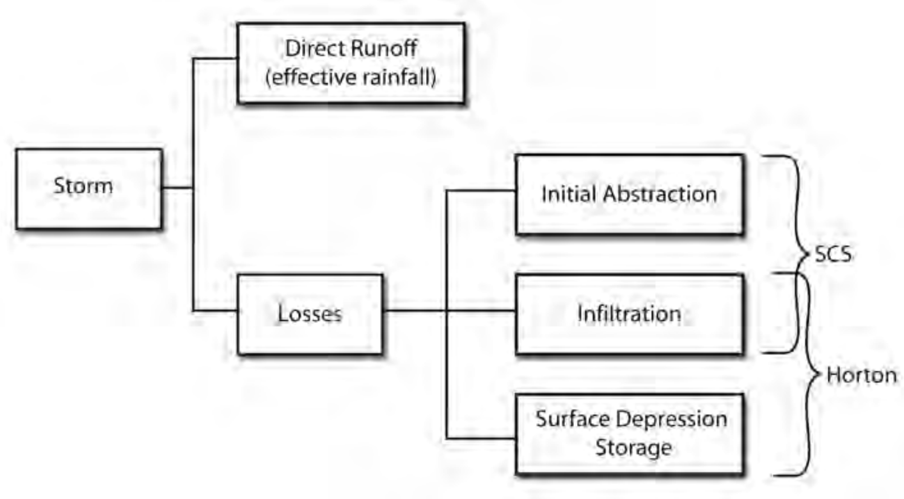


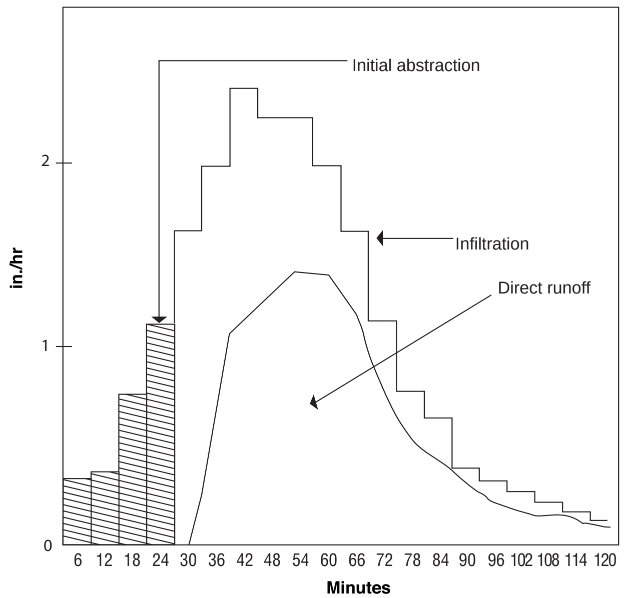
Figure 3.8 Conceptual components of rainfall.

The incident storm is assumed to be represented by a second quartile Huff curve with a total depth of 2 inches and a duration of 120 minutes.

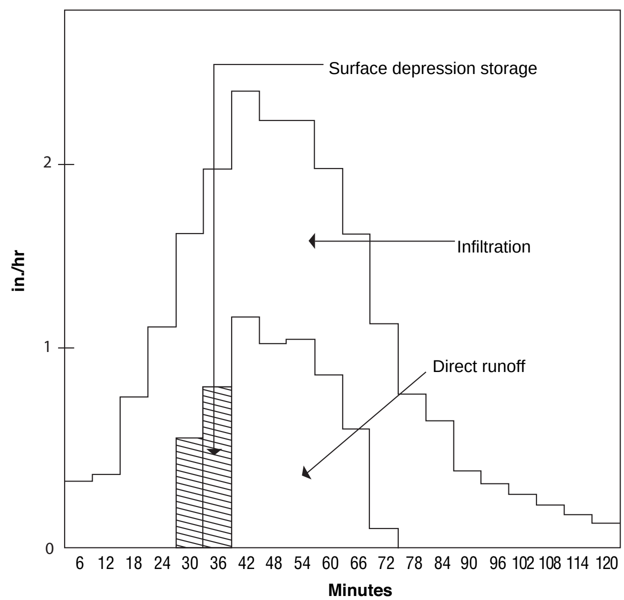
In one case the SCS Method is used with the initial abstraction set at an absolute value of I_a = 0.24 in. The curve number used is 87.6. Figure 3.9 shows that no runoff occurs until approximately 30 minutes have elapsed, at which time the rainfall has satisfied the initial abstraction. From that point, however, the runoff, although small, is finite and continues to be so right to the end of the storm.

The Horton case is tested using values of f_o = 1.2 in./hr, f_c = 0.4 in./hr, K = 0.25 hour, and a surface depression storage depth of 0.2 inches. These values have been found to give the same volumetric runoff coefficient as the SCS parameters. Figure 3.10 shows that infiltration commences immediately and absorbs all of the rainfall until approximately 30 minutes have elapsed. The initial excess surface water has to fill the surface depression storage which delays the commencement of runoff for a further 13 minutes. After 72 minutes the rainfall intensity is less than f_c and runoff is effectively stopped at that time.

It will be found that the effective rainfall hyetograph generated using the Horton equation has more leading and trailing "zero" elements, so that the effective hyetograph is shorter but more intense than that produced using the SCS Method.



■ **Figure 3.9** SCS Method with I_a = 0.24 in. and CN = 87.6.



■ **Figure 3.10** Horton equation with f_o = 1.18 in./hr, f_c = 0.39 in./hr, K = 0.25, and surface depression storage = 0.2 inches.

ESTABLISHING THE TIME OF CONCENTRATION

Apart from the area and the percentage of impervious surface, one of the most important characteristics of a catchment is the time that must elapse until the entire area is contributing to runoff at the outflow point. This is generally called the time of concentration (T_c). This time is comprised of two components:

1) The time for overland flow to occur from a point on the perimeter of the catchment to a natural or artificial drainage conduit or channel.
2) The travel time in the conduit or channel to the outflow point of the catchment.

In storm sewer design, the time of concentration may be defined as the inlet time plus travel time. Inlet times used in sewer design generally vary from 5 to 20 minutes, with the channel flow time being determined from pipe flow equations.

Factors Affecting Time of Concentration

The time taken for overland flow to reach a conduit or channel depends on a number of factors:

a) Overland flow length (L). This should be measured along the line of longest slope from the extremity of the catchment to a drainage conduit or channel. Long lengths result in long travel times.
b) Average surface slope (S). Since T_c is inversely proportional to S, care must be exercised in estimating an average value for the surface slope.
c) Surface roughness. In general, rough surfaces result in longer travel times and smooth surfaces result in shorter travel times. Therefore, if a Manning equation is used to estimate the velocity of overland flow, T_c will be proportional to the Manning roughness factor (n).
d) Depth of overland flow (y). Very shallow surface flows move more slowly than deeper flows. However, the depth of flow is not a characteristic of the catchment alone but depends on the intensity of the effective rainfall and surface moisture excess.

Several methods of estimating the time of concentration are described below. Since it is clear that this parameter has a strong influence on the shape of the runoff hydrograph, it is desirable to compare the value to that obtained from observation, if possible. In situations where sufficient historical data is not available, it may help to compare the results obtained by two or more methods. The impact on the resultant hydrograph, due to using different methods for establishing the time of concentration, should then be assessed.

The Kirpich Formula

This empirical formula relates T_c to the length and average slope of the basin by the equation:

$$T_c = 0.000128\ L^{0.77}\ S^{-0.385} \text{ (See Figure 3.11)}$$

where:
- T_c = time of concentration (hours)
- L = maximum length of water travel (ft)
- S = surface slope, given by H/L (ft/ft)
- H = difference in elevation between the most remote point on the basin and the outlet (ft)

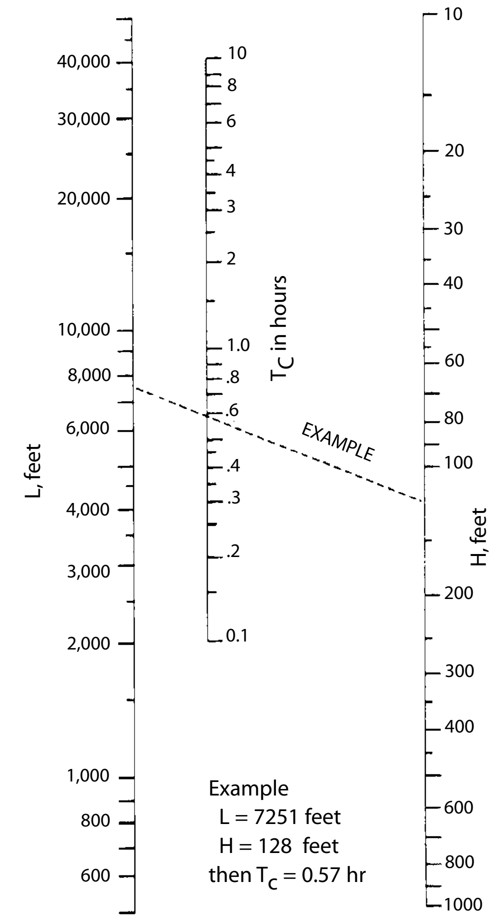


Figure 3.11 T_c nomograph using the Kirpich formula.

From the definition of L and S it is clear that the Kirpich formula combines both the overland flow, or entry time, and the travel time in the channel or conduit. It is, therefore, particularly important that in estimating the drop (H), the slope (S) and ultimately the time of concentration (T_c), an allowance, if applicable, be made for the inlet travel time.

The Kirpich formula is normally used for natural basins with well defined routes for overland flow along bare earth or mowed grass roadside channels. For overland flow on grassed surfaces, the value of T_c obtained should be doubled. For overland flow in concrete channels, a multiplier of 0.2 should be used.

For large watersheds, where the storage capacity of the basin is significant, the Kirpich formula tends to significantly underestimate T_c.

The Uplands Method

When calculating travel times for overland flow in watersheds with a variety of land covers, the Uplands Method may be used. This method relates the time of concentration to the basin slope, basin length and type of ground cover. Times are calculated for individual areas, with their summation giving the total travel time.

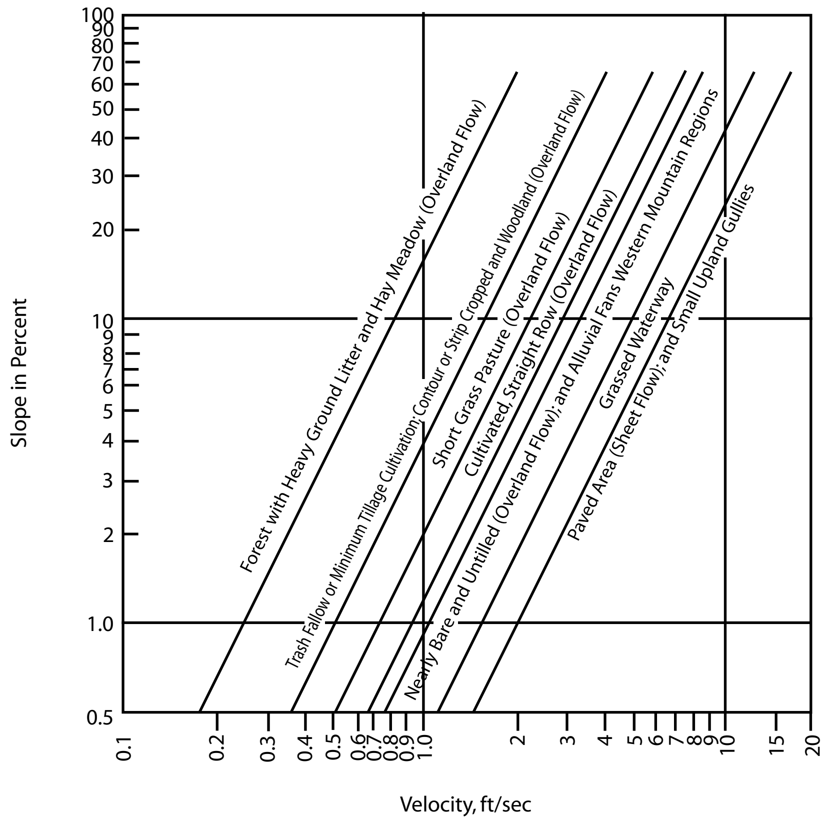
A velocity is derived using the $V/S^{0.5}$ values from Table 3.9 and a known slope. The time of concentration is obtained by dividing the length by the velocity.

Table 3.9

$V/S^{0.5}$ Relationship for Various Land Covers

Land Cover	$V/S^{0.5}$ (ft/s)
Forest with Heavy Ground Litter, Hay Meadow (overland flow)	2.0
Trash Fallow or Minimum Tillage Cultivation; Contour, Strip Cropped, Woodland (overland flow)	5.0
Short Grass Pasture (overland flow)	7.5
Cultivated, Straight Row (overland flow)	9.0
Nearly Bare and Untilled (overland flow) or Alluvial Fans in Western Mountain Regions	10.0
Grassed Waterway	15.0
Paved Areas (sheet flow); Small Upland Gullies	20.0

A graphical solution can be obtained from Figure 3.12. However, it should be noted that the graph is simply a log-log plot of the values of $V/S^{0.5}$ given in Table 3.9.



■ **Figure 3.12** Velocities for Uplands Method for estimating travel time for overland flow.

The Kinematic Wave Method

The two methods described above have the advantage of being quite straightforward and may be used for either simple or more complex methods of determining the runoff. Apart from the empirical nature of the equations, the methods assume that the time of concentration is independent of the depth of overland flow, or more generally, the magnitude of the input. A method in common use, which is more physically based and which also reflects the dependence of T_c on the intensity of the effective rainfall, is the Kinematic Wave Method.

The method was proposed to analyze the kinematic wave resulting from rainfall of uniform intensity on an impermeable plane surface or rectangular area. The resulting equation is as follows:

$$T_c = 0.116\ (L \bullet n/S)^{0.6}\ i_{eff}^{\ -0.4}$$

where: T_c = time of concentration (hr)
L = length of overland flow (ft)
n = Manning's roughness coefficient
S = average slope of overland flow (ft/ft)
i_{eff} = effective rainfall intensity (in./hr)

Other Methods

Other methods have been developed to determine T_c for specific geographic regions or basin types. These methods are often incorporated into an overall procedure for determining the runoff hydrograph. Before using any method, the user should ensure that the basis on which the time of concentration is determined is appropriate for the area under consideration.

DETERMINATION OF THE RUNOFF HYDROGRAPH

The following sections outline alternative methods for generating the runoff hydrograph, which is the relationship of discharge over time. Emphasis will be given to establishing the hydrograph for single storm events. Methods for estimating flow for urban and rural conditions are given.

Irrespective of the method used, the results should be compared to historical values wherever possible. In many cases, a calibration/validation exercise will aid in the selection of the most appropriate method.

All of the methods described could be carried out using hand calculations. However, for all but the simplest cases the exercise would be very laborious. Furthermore, access to computers and computer models is very economical. For these reasons, emphasis will be placed on describing the basis of each method and the relevant parameters. A subsequent section will relate the methods to several computer models.

Rainfall-runoff models may be grouped into two general classifications, which are illustrated in Figure 3.13.

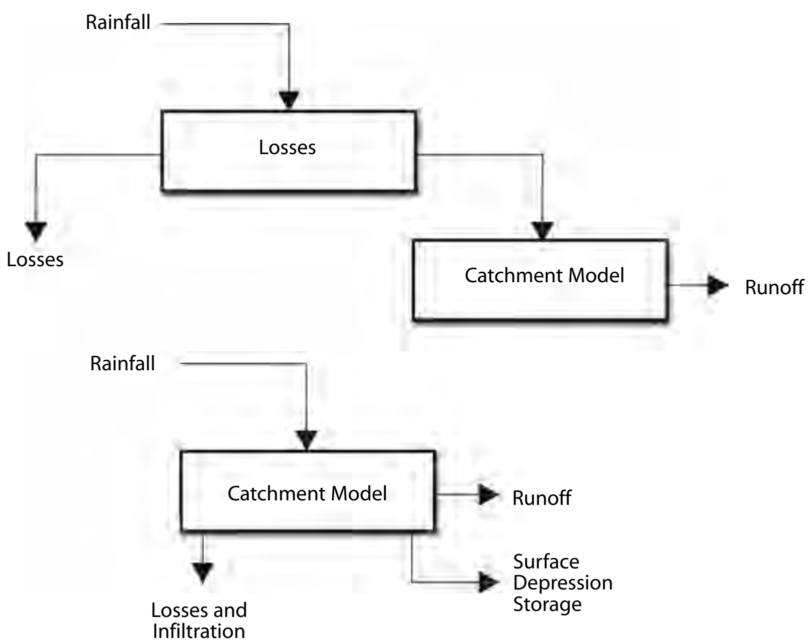


Figure 3.13 Classification of rainfall-runoff models: Effective Rainfall (top) and Surface Water Budget (bottom).

One approach uses the concept of effective rainfall, in which a loss model is assumed that divides the rainfall intensity into losses (initial infiltration and depression storage) and effective rainfall. The effective rainfall hyetograph is then used as input to a catchment model to produce a runoff hydrograph. It follows from this approach that infiltration must stop at the end of the storm.

The alternative approach employs a surface water budget in which the infiltration or loss mechanism is incorporated into the catchment model. In this method, the storm rainfall is used as input and the estimation of infiltration and other losses is an integral part of the runoff calculation. This approach implies that infiltration will continue as long as there is excess water on the surface. Clearly, this may continue after the rainfall ends.

SCS Unit Hydrograph Method

A unit hydrograph represents the runoff distribution over time for one unit of rainfall excess over a drainage area for a specified period of time. This method assumes that the ordinates of flow are proportional to the volume of runoff from any storm of the same duration. Therefore, it is possible to derive unit hydrographs for various rainfall blocks by convoluting the unit hydrograph with the effective rainfall distribution. The unit hydrograph theory is based on the following assumptions:

1. For a given watershed, runoff-producing storms of equal duration will produce surface runoff hydrographs with approximately equivalent time bases, regardless of the intensity of the rain.
2. For a given watershed, the magnitude of the ordinates representing the instantaneous discharge from an area will be proportional to the volumes of surface runoff produced by storms of equal duration.
3. For a given watershed, the time distribution of runoff from a given storm period is independent of precipitation from antecedent or subsequent storm periods.

The U.S. Natural Resource Conservation Service (formerly Soil Conservation Service), based on the analysis of a large number of hydrographs, proposed a unit hydrograph that only requires an estimate of the time to peak (t_p). Two versions of this unit hydrograph were suggested; one being curvilinear in shape, while the other is a simple asymmetric triangle as shown in Figure 3.14. The SCS has indicated that the two hydrographs give very similar results as long as the time increment is not greater than 0.20 •T_c.

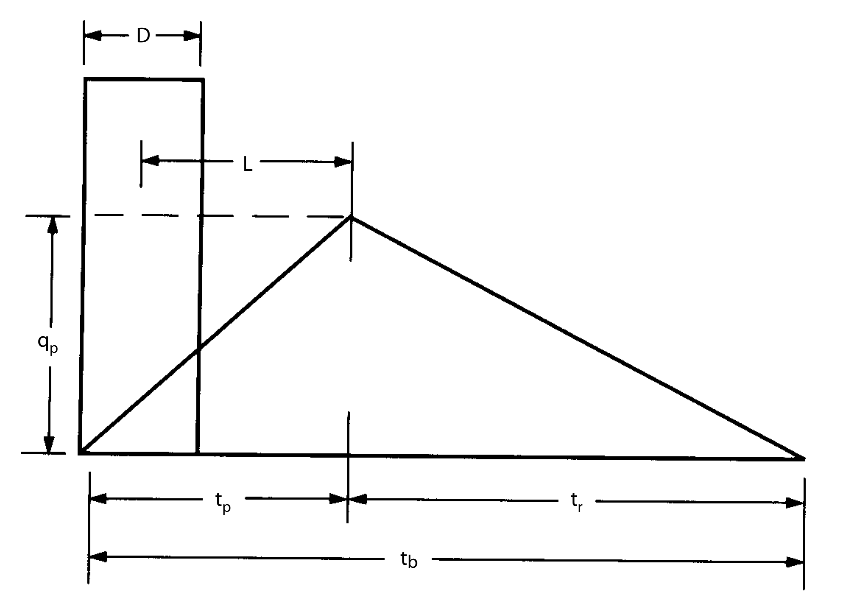


Figure 3.14 SCS triangular unit hydrograph.

The following parameters must be determined to define the triangular unit hydrograph; the time to peak (t_p), the peak discharge corresponding to 1 inch of runoff (q_p), and the base time of the hydrograph (t_b).

Once these parameters are determined, the unit hydrograph can be applied to a runoff depth or to a series of runoff depths. When applied to a series of runoff depths, sub-hydrographs are developed for each and summed to provide an overall hydrograph. A series of runoff depths, for instance, may be a sequence of runoff depths such as those values obtained from a hyetograph where excess rainfall is that portion of the rainfall that is runoff, calculated as the rainfall adjusted to account for retention losses.

The lag time (L) is the delay between the center of the excess rainfall period (D) and the peak of the runoff (t_p). The SCS has suggested that the lag time, for an average watershed and fairly uniform runoff, can be approximated by:

$$L \approx 0.6\ T_c$$

The estimate of the time to peak (t_p) is therefore affected by the time of concentration (T_c) and the excess rainfall period (D). It is calculated using the relationship:

$$t_p = 0.5\ D + 0.6\ T_c$$

where T_c may be determined by any acceptable method such as those described in the previous section.

For a series of runoff depths, where the timestep used is Δt, the parameter D should be replaced by Δt in the above equation, so that it becomes:

$$t_p = 0.5\ \Delta t + 0.6\ T_c$$

The duration of the recession limb of the hydrograph is assumed to be $t_r = (5/3)\ t_p$ so that the time base is given by $t_b = (8/3)\ t_p$.

The ordinates of the unit hydrograph are expressed in units of discharge per unit depth of runoff. In terms of the notation used in Figure 3.14:

$$q_p = 484\ A/t_p$$

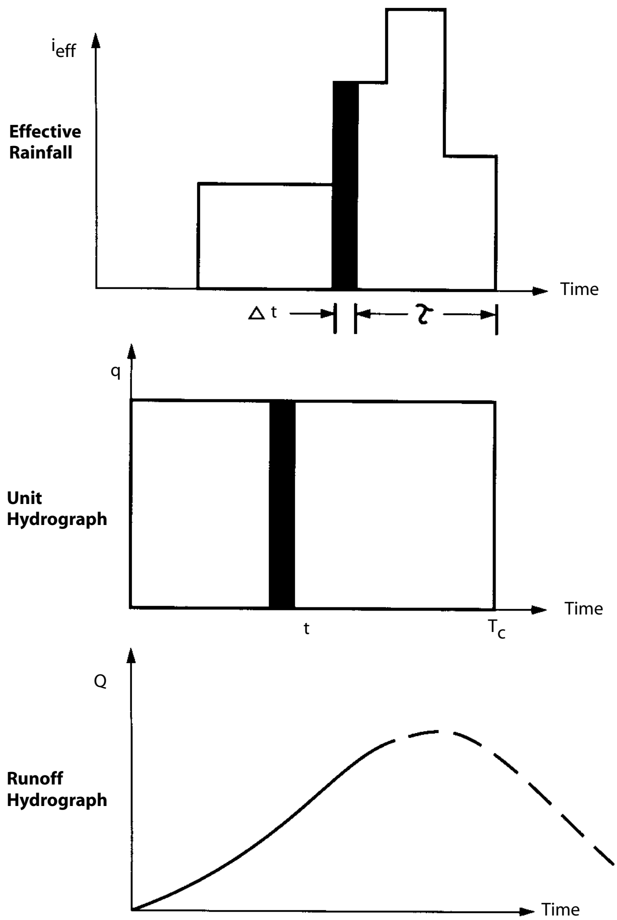
where: q_p = peak discharge, ft^3/s per inch of runoff
A = catchment area, sq. mi.
t_p = time to peak, hours

The numerical constant in the above equation is a measure of the watershed characteristics. This value varies between about 300 for flat marshy catchments and 600 for steep flashy catchments. A value of 484 is recommended by the SCS for average watersheds.

From the above equation it can be seen that the time to peak (t_p), and therefore the peak discharge of the unit hydrograph (q_p), is affected by the value of the excess rainfall period (D) and, in the case of a series of runoff depths, the timestep used (Δt). Values of D or Δt in excess of 0.25 t_p should not be used as this can lead to the underestimation of the peak runoff.

Rectangular Unit Hydrograph Method

An alternative option to the triangular distribution used in the SCS Method is the rectangular unit hydrograph. Figure 3.15 illustrates the concept of convoluting the effective rainfall with a rectangular unit hydrograph. The ordinate of the unit hydrograph is defined as the area of the unit hydrograph divided by the time of concentration (T_c).



■ **Figure 3.15** Convolution process using a rectangular unit hydrograph.

The Rational Method is often used for a rough estimate of the peak flow. This method, which assumes the peak flow occurs when the entire catchment surface is contributing to runoff, may be simulated using a rectangular unit hydrograph. The effective rainfall hydrograph is reduced to a simple rectangular function and i_{eff} = k • C • i. The effective rainfall, with duration t_d, is convoluted with a rectangular unit hydrograph that has a base equal to the time of concentration (T_c). If t_d is made equal to T_c, the resultant runoff hydrograph will be symmetrical and triangular in shape with a peak flow given by Q = k•C•i•A and a time base of t_b = 2 T_c. If the rainfall duration (t_d) is not equal to T_c, then the resultant runoff hydrograph is trapezoidal in shape with a time base of $t_b = t_d + T_c$ and a peak flow given by the following

$$Q = k \bullet C \bullet i \bullet A \,(t_d / T_c) \qquad \text{for } td \le T_c$$

$$Q = k \bullet C \bullet i \bullet A \qquad \text{for } td > T_c$$

This approach makes no allowance for the storage effect due to the depth of overland flow and results in an “instantaneous” runoff hydrograph. This may be appropriate for impervious surfaces in which surface depression storage is negligible, but for pervious or more irregular surfaces it may be necessary to route the instantaneous hydrograph through a hypothetical reservoir in order to more closely represent the runoff hydrograph.

Linear Reservoir Method

Pederson suggested a more complex response function in which the shape of the unit hydrograph is assumed to be the same as the response of a single linear reservoir to an inflow having a rectangular shape and duration Δt. A linear reservoir is one in which the storage (S) is linearly related to the outflow (Q) by the formula:

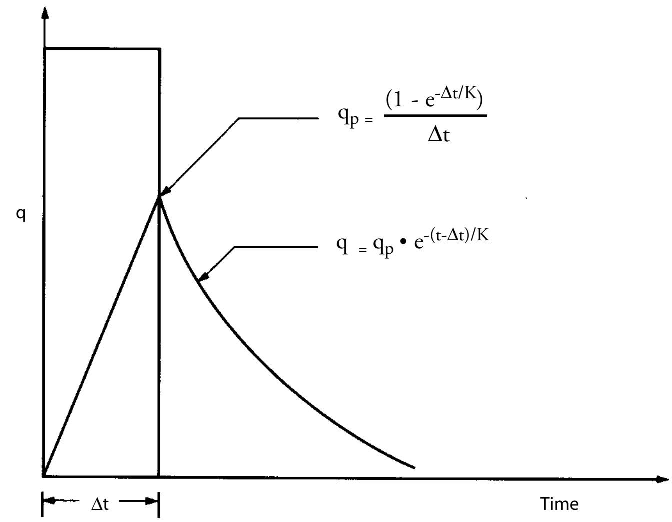
$$S = K \bullet Q$$

where: K = the reservoir lag or storage coefficient (hours)

In Pederson’s method, the value of K is taken to be 0.5 T_c where T_c is computed from the kinematic wave equation in which the rainfall intensity used is the maximum for the storm being modeled. The use of i_{max} is justified since this intensity tends to dominate the subsequent runoff hydrograph. The resulting unit hydrograph is illustrated in Figure 3.16 and comprises a steeply rising limb, which reaches a maximum at time t = Δt, followed by an exponential recession limb. The two curves can be described by the following

$$q_p = \frac{(1 - e^{-\Delta t/K})}{\Delta t} \quad \text{at } t = \Delta t$$

$$q = q_p \bullet e^{-(t-\Delta t)/K} \quad \text{for } t > \Delta t$$



■ **Figure 3.16** The single linear reservoir.

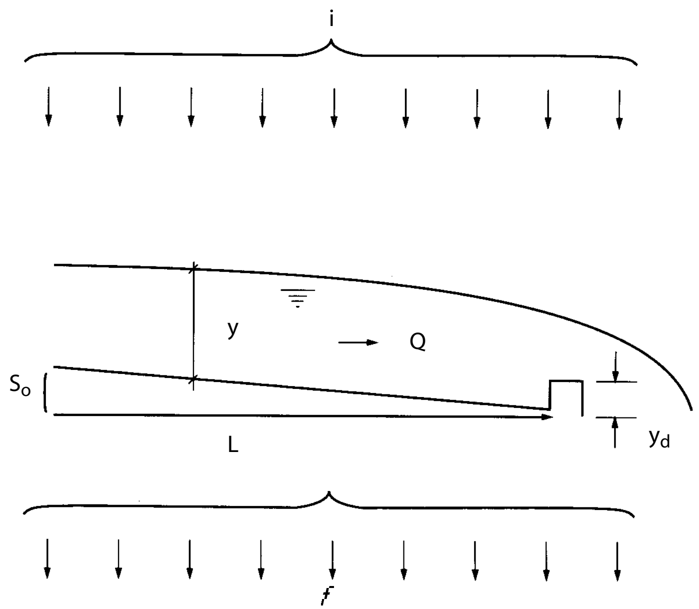
An important feature of the method is that the unit hydrograph always has a time to peak of Δt and is incapable of reflecting different response times as a function of catchment length, slope or roughness. It follows that the peak of the runoff hydrograph will usually be close to the time of peak rainfall intensity, irrespective of the catchment characteristics.

SWMM Runoff Algorithm

The Storm Water Management Model was originally developed for the U.S. Environmental Protection Agency in 1971. Since then it has been expanded and improved by the EPA and many other agencies and companies. In particular, the capability for continuous simulation has been included (in addition to the original ability to handle single event simulation), quality as well as quantity is simulated, and snow-melt routines are included in some versions.

The model is intended for use in urban or partly urban catchments. It comprises five main "blocks" of code in addition to an Executive Block or supervisory calling program. Following is a description of the basic algorithm of the Runoff Block, which is used to generate the runoff hydrograph in the drainage system based on a rainfall hyetograph, Antecedent Moisture Conditions, land use and topography.

The method differs from those described previously in that it does not use the concept of effective rainfall, but employs a surface water budget approach. In that approach, rainfall, infiltration, depression storage and runoff are all considered as processes occurring simultaneously at the land surface. The interaction of these inputs and outputs may be visualized with reference to Figure 3.17.



■ **Figure 3.17** Representation of the SWMM/Runoff algorithm.

Treating each sub-catchment as an idealized, rectangular plane surface having a breadth (B) and length (L), the continuity or mass balance equation at the land surface is given by:

Inflow = (Infiltration + Outflow) + Rate of Surface Ponding

This may be expressed as

$$i \bullet L \bullet B = (f \bullet L \bullet B + Q) + L \bullet B \bullet (\Delta y/\Delta t)$$

where: i = rainfall intensity
f = infiltration rate
Q = outflow
y = depth of flow over the entire surface

The depth of flow (y) is computed using the Manning equation, taking into account the depth of surface depression storage (y_d) which is also assumed to be uniform over the entire surface. The dynamic equation is given by:

$$Q = B\ (1/n)\ (y - y_d)^{5/3}\ S^{1/2}$$

where: n = Manning's roughness coefficient for overland flow
S = average slope of the overland flow surface

The infiltration rate (f) must be computed using a method such as the 'moving curve' Horton equation or the Green-Ampt model. Infiltration is assumed to occur as long as excess surface moisture is available from rainfall, depression storage or finite overland flow.

It is important to note that the value of Manning's "n" used for overland flow is somewhat artificial (for example, in the range of 0.1 to 0.4) and does not represent a value that might be used for channel flow calculation.

Various methods can be used for the simultaneous solution of the continuity and dynamic equations. One method is to combine the equations into one nondifferential equation in which the depth (y) is the unknown. Once the depth is determined (for instance, by an interactive scheme such as the Newton-Raphson Method) the outflow (Q) follows.

COMPUTER MODELS

Many computer models have been developed for the simulation of the rainfall/runoff process. Table 3.10 lists several of these models and their capabilities.

Table 3.10

Hydrologic computer models

Model Characteristics	HEC-1	HYMO	HSPF	ILLUDAS	MIDUSS	OTTHYMO	QUALHYMO	SCS TR-20	SCS TR-55	SSARR	STANFORD	STORM	SWMM	USDAHL-74
	Models													
Model Type:														
Single Event	●	●		●	●	●		●	●				●	
Continuous			●				●			●	●	●	●	●
Model Components:														
Infiltration	●	●	●	●	●	●	●	●	●	●	●	●	●	●
Evapotranspiration			●							●	●	●	●	●
Snowmelt	●		●							●	●	●	●	●
Surface Runoff	●	●	●	●	●	●	●	●	●	●	●	●	●	●
Subsurface Flow	●		●				●			●	●			●
Reservoir Routing	●	●	●		●	●	●	●	●	●			●	
Channel Routing	●	●	●	●	●	●	●	●	●	●	●		●	
Water Quality			●				●					●	●	
Application:														
Urban Land Use			●	●	●	●	●		●			●	●	
Rural Land Use	●	●	●		●	●	●	●		●	●	●		●
Ease of Use:														
High	●	●		●	●	●	●	●	●			●		●
Low			●							●	●		●	

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Deep corrugated structural arch during high flow.

Round pipe multiple opening installation.

CHAPTER four

INTRODUCTION

Many millions of dollars are spent annually on culverts, storm drains and subdrains, all vital to the protection of streets, highways and railroads. If inadequately sized, they can jeopardize the roadway and cause excessive property damage and loss of life. Over design means extravagance. Engineering can find an economical solution.

Topography, soil and climate are extremely variable, so drainage sites should be designed individually from reasonably adequate data for each particular site. In addition, the designer is advised to consult with those responsible for maintaining drainage structures in the area. One highway engineer comments:

> With the exception of the riding qualities of the traveled way, no other single item requires as much attention on the part of maintenance personnel as highway culverts. Many of the problems of culvert maintenance stem from the fact that designers in all too many instances consider that culverts will be required to transport only clear water. This is a condition hardly ever realized in practice, and in many instances storm waters may be carrying as much as 50 percent detrimental material. A rapid change in grade line at the culvert entrance can cause complete blockage of the culvert. This results in overflow across the highway and in some cases, especially where high fills are involved, the intense static pressure results in loss of the embankment.

HYDRAULICS OF OPEN DRAINAGE CHANNELS

General

Before designing culverts and other drainage structures, one should consider the design of ditches, gutters, chutes, median swales and other channels leading to these structures. (See Figure 4.1).

Rainfall and runoff, once calculated, are followed by the design of suitable channels to handle the peak discharge with minimum erosion, maintenance and hazard to traffic. The AASHTO publication "A Policy on Geometric Design of Highways and Streets" states: "The depth of channels should be sufficient to remove the water without saturation of the pavement subgrade. The depth of water that can be tolerated, particularly on flat channel slopes, depends upon the soil characteristics. In open country, channel side slopes of 5:1 or 6:1 are preferable in order to reduce snow drifts." Systematic maintenance is recognized as essential to any drainage channel. Therefore maintenance should be considered in the design of all channels.

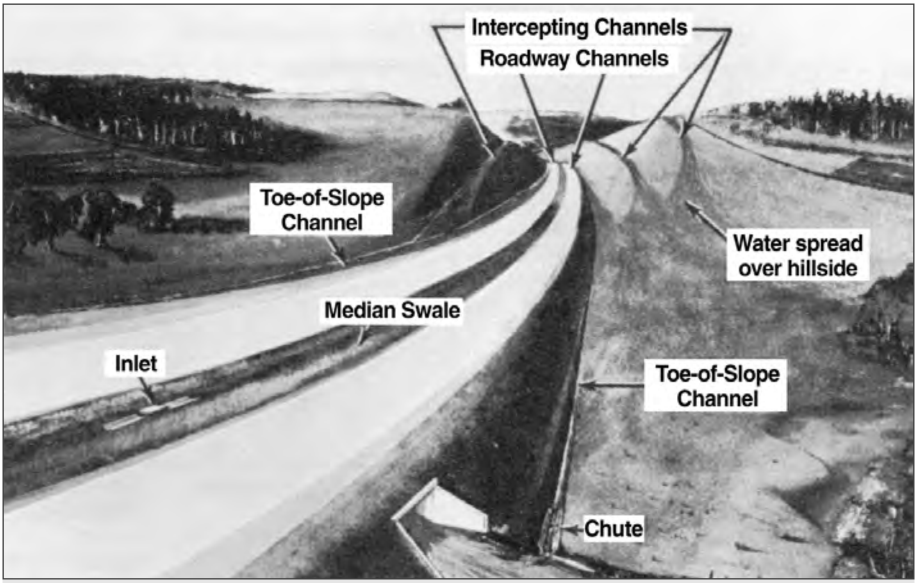


Figure 4.1 Types of roadside drainage channels.

Chezy Equation

Chezy developed a basic hydraulic relationship for determining the flow of water, particularly in open channels. It is as follows:

$$Q = AV$$

if: $$V = c\sqrt{RS}$$

then: $$Q = Ac\sqrt{RS}$$

where:
- Q = discharge, ft^3/s
- A = cross-sectional area of flow, ft^2
- V = mean velocity of flow, ft/s
- c = coefficient of roughness, depending upon the surface over which water is flowing, $ft^{1/2}/s$
- R = hydraulic radius, ft $= \frac{A}{WP}$
- WP = wetted perimeter (length of wetted contact between water and its containing channel), ft
- S = slope, or grade, ft/ft

This fundamental equation is the basis of most capacity formulations.

Manning's Equation

Manning's equation, published in 1890, gives the value of c in the Chezy equation as:

$$c = 1.486 \frac{R^{1/6}}{n}$$

where: n = coefficient of roughness (see Tables 4.1 and 4.2)

Table 4.1

Manning's *n* for constructed channels

Types of channel and description	*n*
1. LINED OR BUILT-UP	
A. Concrete - Trowel Finish	0.013
B. Concrete - Float Finish	0.015
C. Concrete - Unfinished	0.017
D. Gunite - Good Section	0.019
E. Gravel Bottom with sides of:	
1) Formed Concrete	0.020
2) Random Stone in Mortar	0.023
3) Dry Rubble or Rip Rap	0.033
2. EXCAVATED OR DREDGED - EARTH	
A. Straight and Uniform	
1) Clean, Recently Completed	0.018
2) Clean, After Weathering	0.022
3) Gravel, Uniform Section, Clean	0.025
4) With Short Grass, Few Weeds	0.027
B. Winding and Sluggish	
1) No Vegetation	0.025
2) Grass, Some Weeds	0.030
3) Dense Weeds, Deep Channels	0.035
4) Earth Bottom and Rubble Sides	0.030
5) Stony Bottom and Weedy Banks	0.035
6) Cobble Bottom and Clean Sides	0.040
3. CHANNELS NOT MAINTAINED, WEEDS & BRUSH UNCUT	
A. Dense Weeds, High as Flow Depth	0.080
B. Clean Bottom, Brush on Sides	0.050
C. Same, Highest Stage of Flow	0.070
D. Dense Brush, High Stage	0.100

Table 4.2

Manning's *n* for natural stream channels
Surface width at flood stage less than 100 ft.

1. Fairly regular section:	
a. Some grass and weeds, little or no brush	0.030–0.035
b. Dense growth of weeds, depth of flow materially greater than weed height	0.035–0.05
c. Some weeds, light brush on banks.	0.035–0.05
d. Some weeds, heavy brush on banks	0.05–0.07
e. Some weeds, dense willows on banks.	0.06–0.08
f. For trees within channel, with branches submerged at high stage, increase all above values by	0.01–0.02
2. Irregular sections, with pools, slight channel meander; increase values given above about	0.01–0.02
3. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage:	
a. Bottom of gravel, cobbles, and few boulders	0.04–0.05
b. Bottom of cobbles, with large boulders	0.05–0.07

The complete Manning equation is:

$$V = 1.486 \frac{R^{2/3}S^{1/2}}{n}$$

Combining this with the Chezy Equation results in the equation:

$$Q = 1.486 \frac{AR^{2/3}S^{1/2}}{n}$$

In many calculations, it is convenient to group the channel cross section properties in one term called conveyance, K, so that:

$$K = 1.486 \frac{AR^{2/3}}{n}$$

then:

$$Q = KS^{1/2}$$

Uniform flow of clean water in a straight unobstructed channel would be an ideal condition, but it is rarely found. Manning's equation gives reliable results if the channel cross section, roughness, and slope are fairly constant over a sufficient distance to establish uniform flow.

The Use of Charts and Tables

While design charts for open-channel flow reduce computational effort, they cannot replace engineering judgment and a knowledge of the hydraulics of open-channel flow and flow through conduits with a free water surface.

Design charts contain the channel properties (area and hydraulic radius) of many channel sections and tables of velocity for various combinations of slope and hydraulic radius. Their use is explained in the following examples:

Example 1

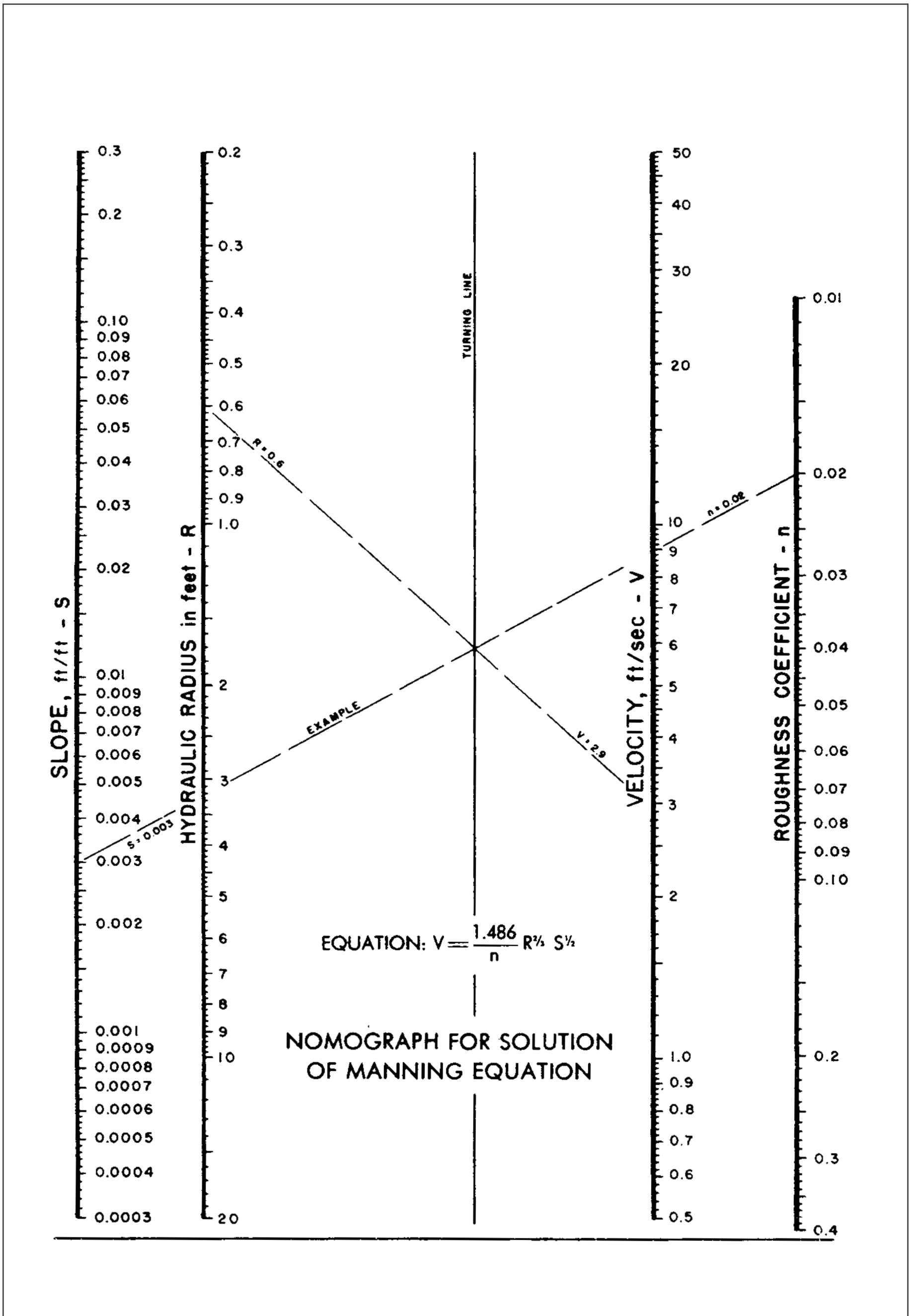
Given: A trapezoidal channel of straight alignment and uniform cross section in earth with a bottom width of 2 feet, side slopes at 1:1, a channel slope of 0.003 ft/ft, and a normal depth of water of 1 foot.

Find: Velocity and discharge.

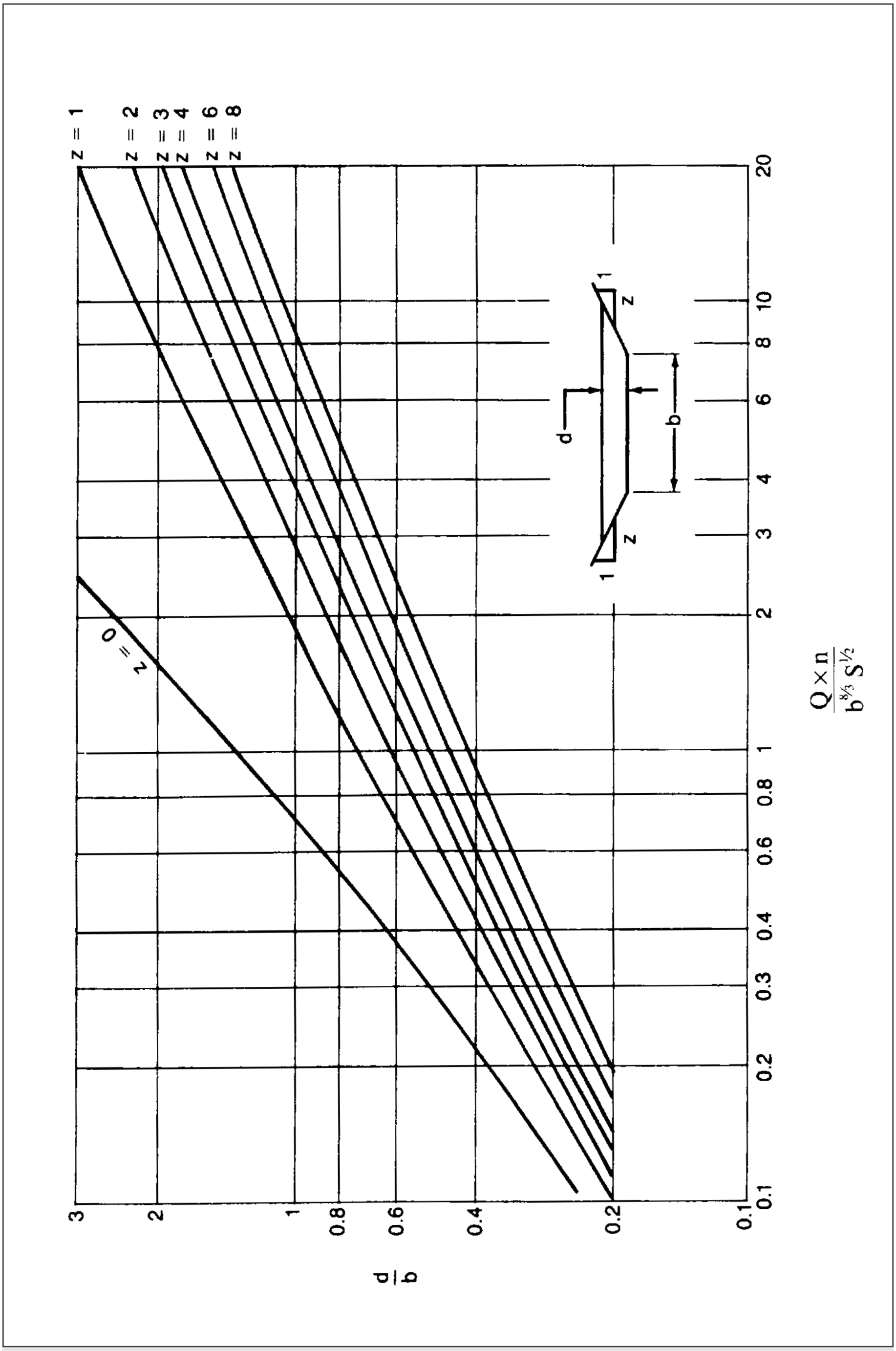
Solution:

1. Based on Table 4.1, for an excavated channel in ordinary earth, n is taken as 0.02.
2. Cross-sectional area, A, is 3 ft^2 [1 * (2 + 1 * 1)].
3. Wetted perimeter, WP, is 4.83 ft [2 + 2 *(1 * $(1^2+1^2)^{1/2}$)].
4. Hydraulic radius, R, is 0.62 ft [3 / 4.83].
5. Using the nomograph in Figure 4.2, lay a straight edge between the outer scales at the values of S = 0.003 and n = 0.02. Mark where the straight edge intersects the turning line.
6. Place the straight edge to line up the point on the turning line and the hydraulic radius of 0.62 ft.
7. Read the velocity, V, of 2.9 ft/s on the velocity scale.
8. Discharge, Q, is 8.7 ft^3/s[3 * 2.9].

Figure 4.3 provides the means to calculate a trapezoidal channel capacity for a specific bottom width, channel slope, side slope, n value and a variety of flow depths. For a given drainage project, these variables are either known or determined using known site parameters through trial and error. The flow rate, Q, can then be calculated.



■ **Figure 4.2** Nomograph for solution of Manning's equation.



■ **Figure 4.3** Capacity of trapezoidal channel.

Example 2

Given: Bottom width, b = 20 ft
Side slopes @ 2:1, z = 2
Roughness coefficient, n = 0.030
(from Table 4.2 for grass and weeds, no brush)
Channel slope, S = 0.002 ft/ft
Depth to bottom width ratio, $^d/_b$ = 0.6 (flood stage depth)

Find: Depth of flow, d, and flow rate, Q.

Solution:

1. Depth, d = 12 ft [0.6 * 20]
2. From Figure 4.3:

$$\frac{Q \bullet n}{b^{8/3}\, S^{1/2}} = 0.92$$

3. So:

$$\frac{Q\,(0.030)}{20^{8/3}\,(0.002)^{1/2}} = 0.92$$

4. And: $Q = 4042\ \text{ft}^3/\text{s}$

If the resulting design is not satisfactory, the channel parameters are adjusted and the design calculations are repeated.

Safe Velocities

The ideal situation is one where the velocity will cause neither silt deposition nor erosion. For the design of a channel, the approximate grade can be determined from a topographic map, from the plan profiles or from both.

To prevent the deposition of sediment, the minimum gradient for earth and grass-lined channels should be about 0.5 percent and that for smooth paved channels about 0.35 percent.

Convenient guidelines for permissible velocities are provided in Tables 4.3 and 4.4. More comprehensive design data may be found in HEC 15, Design of Stable Channels with Flexible Linings, U.S. Federal Highway Administration (FHWA).

Table 4.3

Comparison of water velocity limits and tractive force values for the design of stable channels

Material	*n*	For Clear Water Velocity ft/sec	For Clear Water Tractive* Force lb/ft²	Water Transporting Colloidal Silts Velocity ft/sec	Water Transporting Colloidal Silts Tractive* Force lb/ft²
Fine sand colloidal	0.020	1.50	0.027	2.50	0.075
Sandy loam noncolloidal	0.020	1.75	0.037	2.50	0.075
Silt loam noncolloidal	0.020	2.00	0.048	3.00	0.11
Alluvial silts noncolloidal	0.020	2.00	0.048	3.50	0.15
Ordinary firm loam	0.020	2.50	0.075	3.50	0.15
Volcanic ash	0.020	2.50	0.075	3.50	0.15
Stiff clay very colloidal	0.025	3.75	0.26	5.00	0.46
Alluvial silts colloidal	0.025	3.75	0.26	5.00	0.46
Shales and hardpans	0.025	6.00	0.67	6.00	0.67
Fine gravel	0.020	2.50	0.075	5.00	0.32
Graded loam to cobbles when non-colloidal	0.030	3.75	0.38	5.00	0.66
Graded silts to cobbles when colloidal	0.030	4.00	0.43	5.50	0.80
Coarse gravel noncolloidal	0.025	4.00	0.30	6.00	0.67
Cobbles and shingles	0.035	5.00	0.91	5.50	1.10

* Tractive force or shear is the force which the water exerts on the periphery of a channel due to the motion of the water. The tractive values shown were computed from velocities given by S. Fortier and Fred C. Scobey and the values of *n* shown.

The tractive force values are valid for the given materials regardless of depth. For depths greater than 3 ft, higher velocities can be allowed and still have the same tractive force.

From *U.S. Bureau of Reclamation*, Report No. Hyd-352, 1952, 60 pp.

Channel Protection

Corrugated steel flumes or chutes and pipe spillways are favored solutions for channel protection, especially in wet, unstable or frost susceptible soils. They should be anchored to prevent undue shifting. This will also protect against buoyancy and uplift, which can occur especially when the pipe is empty. Cutoff walls or collars are used to prevent undermining.

If the mean velocity exceeds the permissible velocity for the particular type of soil, the channel should be protected from erosion. Grass linings are valuable where grass growth can be supported. Ditch bottoms may be sodded or seeded with the aid of temporary quick growing grasses, mulches or erosion control blankets. Grass may also be used in combination with other more rigid types of linings, where the grass is on the upper bank slopes and the rigid lining is on the channel bottom. Linings may consist of stone which is dumped, hand placed or grouted, preferably laid on a filter blanket of gravel or crushed stone and a geotextile.

Table 4.4

Maximum permissible velocities in vegetal-lined channels

Cover Average, Uniform Stand, Well Maintained	Slope Range Percent	Permissible Velocity[a] Erosion Resistant Soils ft/sec	Easily Eroded Soils ft/sec
Bermudagrass	0 - 5 5 - 10 over 10	8 7 6	6 5 4
Buffalograss Kentucky bluegrass Smooth brome Blue grama	0 - 5 5 - 10 over 10	7 6 5	5 4 3
Grass mixture[b]	5 - 10	5 4	4 3
Lespedeza sericea Weeping lovegrass Yellow bluestem Alfalfa Crabgrass	0 - 5	3.5	2.5
Common lespedeza[b] Sudangrass[b]	0 - 5[c]	3.5	2.5

a From "Engineering Field Manual" USDA - Soil Conservation Service, 1979, (now Natural Resource Conservation Service).
b Annuals—used on mild slopes or as temporary protection until permanent covers are established .
c Use on slopes steeper than 5 percent is not recommended.

Asphalt and concrete lined channels are used for steep erodible channels. Ditch checks are an effective means of decreasing the velocity and thereby the erodibility of the soil. High velocities, where water discharges from a channel, must be considered and provisions be made to dissipate the excess energy.

HYDRAULICS OF CULVERTS

Introduction

Culvert design has not yet reached the stage where two or more individuals will always arrive at the same answer, or where actual service performance matches the designer's expectation. The engineer's interpretation of field data and hydrology is often influenced by personal judgement, based on experience in a given locality. However, hydrology and hydraulic research are closing the gap to move the art of culvert design closer to becoming a science.

Up to this point, the design procedure has consisted of (1) collecting field data, (2) compiling facts about the roadway, and (3) making a reasonable estimate of flood discharge. The next step is to design an economical corrugated steel structure to handle the flow, including debris, with minimum damage to the slope or culvert barrel. Treatment of the inlet and outlet ends of the structure must also be considered.

What Makes a Good Culvert?

An ASCE Task Force on Hydraulics of Culverts offers the following recommendations for "Attributes of a Good Highway Culvert":

1. The culvert, appurtenant entrance and outlet structures should properly take care of water, bed load, and floating debris at all stages of flow.

2. It should cause no unnecessary or excessive property damage.

3. Normally, it should provide for transportation of material without detrimental change in flow pattern above and below the structure.

4. It should be designed so that future channel and highway improvement can be made without too much loss or difficulty.

5. It should be designed to function properly after fill has caused settlement.

6. It should not cause objectionable stagnant pools in which mosquitoes may breed.

7. It should be designed to accommodate increased runoff occasioned by anticipated land development.

8. It should be economical to build, hydraulically adequate to handle design discharge, structurally durable and easy to maintain.

9. It should be designed to avoid excessive ponding at the entrance which may cause property damage, accumulation of drift, culvert clogging, saturation of fills, or detrimental upstream deposits of debris.

10. Entrance structures should be designed to screen out material which will not pass through the culvert, reduce entrance losses to a minimum, make use of the velocity of approach in so far as practicable, and by use of transitions and increased slopes, as necessary, facilitate channel flow entering the culvert.

11. The design of the culvert outlet should be effective in re-establishing tolerable non-erosive channel flow within the right-of-way or within a reasonably short distance below the culvert.

12. The outlet should be designed to resist undermining and washout.

13. Energy dissipaters, if used, should be simple, easy to build, economical and reasonably self-cleaning during periods of easy flow.

Design Method

The culvert design process should strive for a balanced result. Pure fluid mechanics should be combined with practical considerations to help assure satisfactory performance under actual field conditions. This includes due consideration of prospective maintenance and the handling of debris.

The California Department of Transportation uses an excellent method of accomplishing this, which has worked well for many years. Other states and agencies have used similar approaches. California culvert design practice establishes the following:

Criteria for Balanced Design

The culvert shall be designed to discharge

a) a 10 year flood without static head at the entrance, and
b) a 100 year flood utilizing the available head at the entrance.

This approach lends itself well to most modern design processes and computer programs such as those published by the U.S. FHWA. It provides a usable rationale for determining a minimum required waterway area. This design method is highly recommended and is followed here in conjunction with FHWA charts.

The permissible height of water at the inlet controls hydraulic design. This should be determined and specified for each site based on the following considerations:

1. Risk of overtopping the embankment and the resulting risk to human life.

2. Potential damage to the roadway, due to saturation of the embankment, and pavement disruption due to freeze-thaw.

3. Traffic interruptions.

4. Damage to adjacent or upstream property, or to the channel or flood plain environment.

5. Intolerable discharge velocities, which can result in scour and erosion.

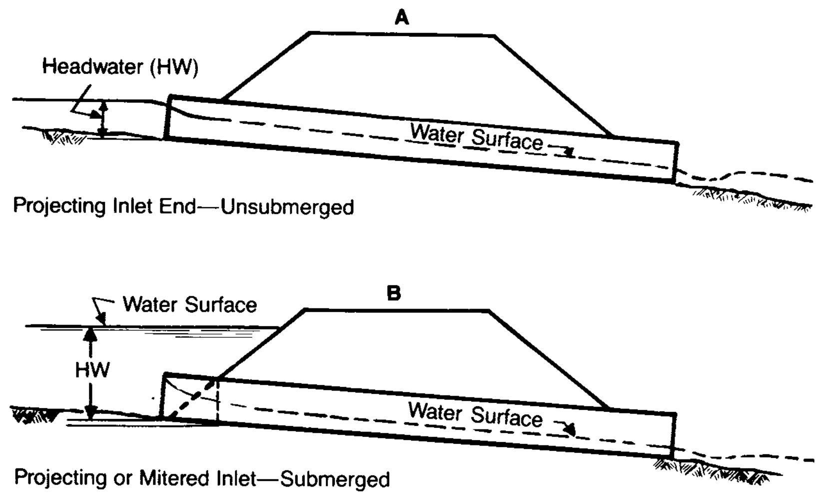
6. Deposition of bed load and/or clogging by debris on recession of flow.

Flow Conditions and Definitions

Culverts considered here are circular pipes and pipe arches with a uniform barrel cross-section throughout.

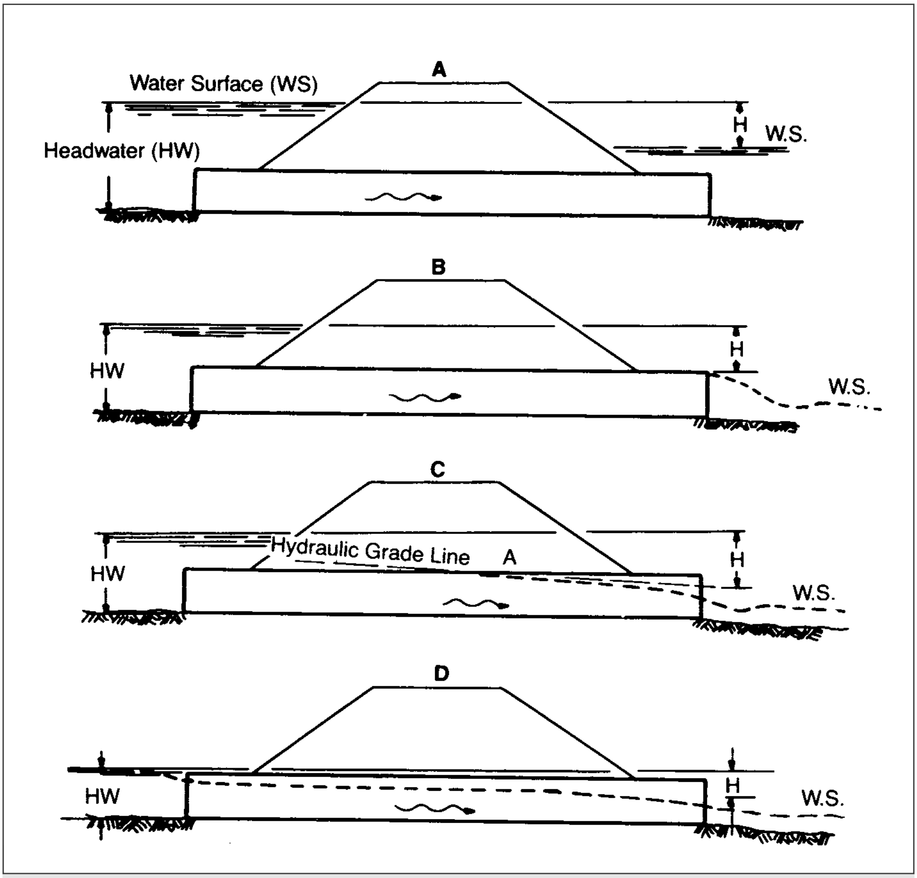
There are two major types of culvert flow conditions:

Inlet Control – A culvert flowing in inlet control is characterized by shallow, high velocity flow categorized as supercritical. Inlet control flow occurs when the culvert barrel is capable of conveying more flow than the inlet will accept. The control section is near the inlet, and the downstream pipe and flow have no impact on the amount of flow through the pipe. Under inlet control, the factors of primary importance are (1) the cross-sectional area of the barrel, (2) the inlet configuration or geometry, and (3) the headwater elevation or the amount of ponding upstream of the inlet (see Figure 4.4). The barrel slope also influences the flow under inlet control, but the effect is small and it can be ignored.



■ **Figure 4.4** Inlet control flow regimes.

Outlet Control – A culvert flowing in outlet control is characterized by relatively deep, lower velocity flow categorized as subcritical. Outlet control flow occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. The control section is at the outlet of the culvert. In addition to the factors considered for inlet control, factors that must be considered for outlet control include (1) the tailwater elevation in the outlet channel, (2) the barrel slope, (3) the barrel roughness, and (4) the length of the barrel (see Figure 4.5).



■ **Figure 4.5** Outlet control flow regimes.

Hydraulics of Culverts in Inlet Control

Inlet control means that the discharge capacity is controlled at the entrance by the headwater depth, cross-sectional area and type of inlet edge. The roughness, length, and outlet conditions are not factors in determining the culvert capacity.

Sketches A and B in Figure 4.4 show unsubmerged and submerged projecting inlets respectively. Inlet control performance is classified by these two regimes (unsubmerged flow and submerged flow) as well as a transition region between them.

Entrance loss depends upon the geometry of the inlet edge and is expressed as a fraction of the velocity head. Research with models and prototype testing have resulted in coefficients for various types of inlets, as shown in Table 4.5 and Figure 4.6.

Table 4.5

Entrance loss coefficients for corrugated steel pipes and pipe arches

Inlet End of Culvert	Entrance Type	Coefficient, k_e
Projecting from fill (no headwall)	1	0.9
Mitered (beveled) to conform to fill slope	2	0.7
Headwall or headwall and wingwalls square-edge	3	0.5
End-Section conforming to fill slope	4	0.5
Headwall rounded edge	5	0.2
Beveled Ring	6	0.25

The model testing and prototype measurements also provide information used to develop equations for unsubmerged and submerged inlet control flow. The transition zone is poorly defined, but it is approximated by plotting the two flow equations and connecting them with a line which is tangent to both curves. These plots, done for a variety of structure sizes, are the basis for constructing the design nomographs included in this design manual.

In the nomographs, the headwater depth (HW) is the vertical distance from the culvert invert (bottom) at the entrance to the energy grade line of the headwater pool. It therefore includes the approach velocity head. The velocity head tends to be relatively small and is often neglected. The resulting headwater depth is therefore conservative and the actual headwater depth would be slightly less than the calculated value. If a more accurate headwater depth is required, the approach velocity head should be subtracted from the headwater depth determined using the nomographs.

Hydraulics of Culverts in Outlet Control

Outlet control means that the discharge capacity is controlled at the outlet by the tailwater depth or critical depth, and it is influenced by such factors as the slope, wall roughness and length of the culvert. The following energy balance equation contains the variables that influence the flow through culverts flowing under outlet control:

$$L \bullet S_o + HW + \frac{V_1^2}{2g} = h_o + H + \frac{V_2^2}{2g}$$

where:
- L = length of culvert, ft
- S_o = slope of barrel, ft/ft
- HW = headwater depth, ft
- V_1 = approach velocity, ft/s
- g = gravitational constant = 32.2 ft/s^2
- h_o = outlet datum, ft
- H = head, ft
- V_2 = downstream velocity, ft/s

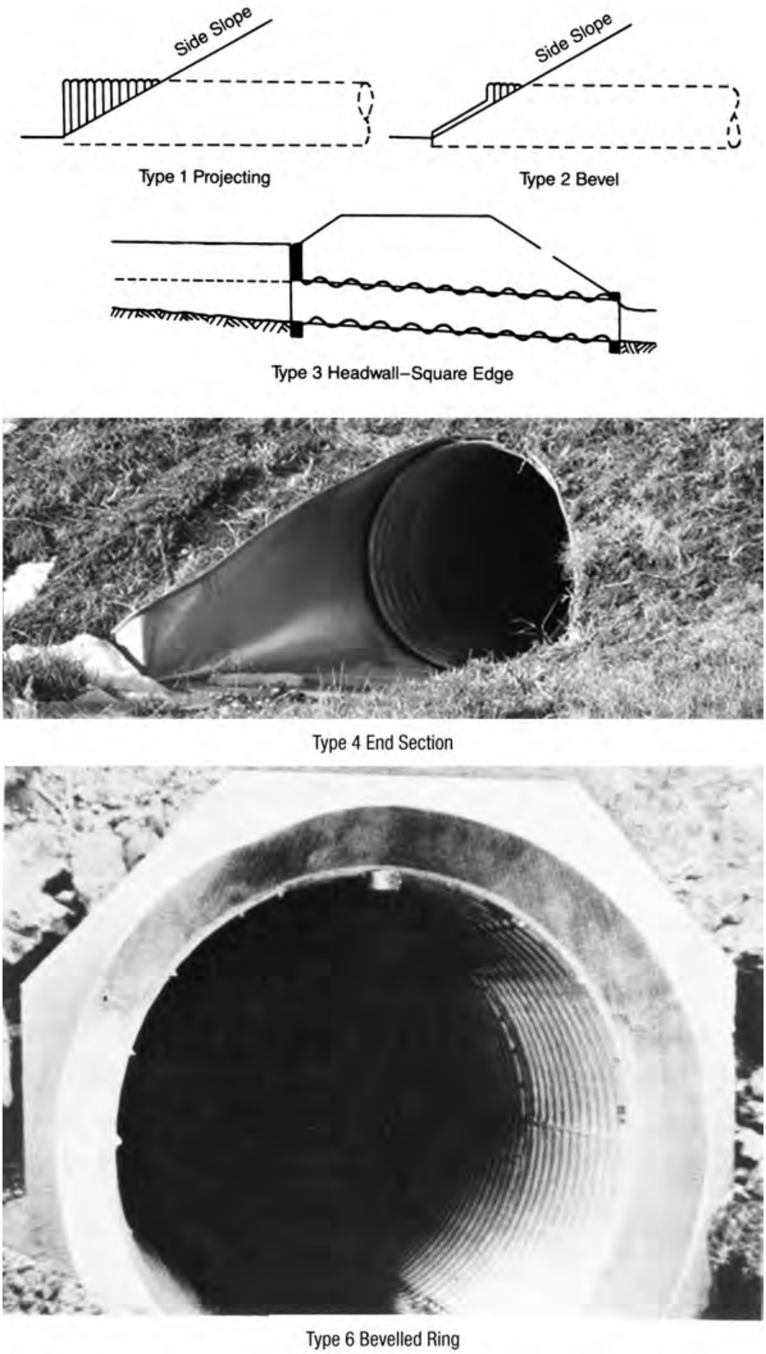


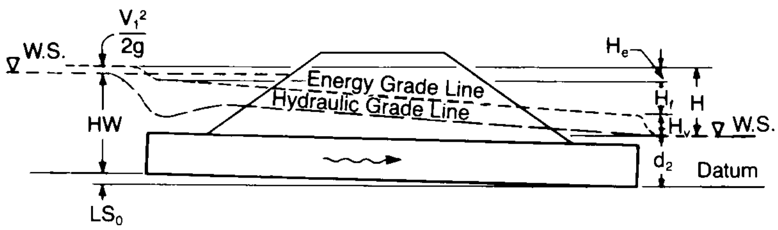
Figure 4.6 Typical entrance types.

The headwater depth (HW) is the vertical distance from the culvert invert at the entrance (where the entrance is that point in the pipe where there is the first full cross-section) to the surface of the headwater pool.

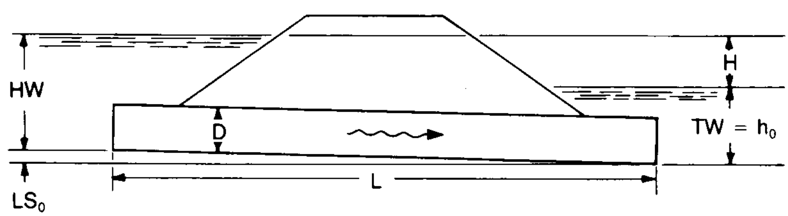
As discussed under inlet control hydraulics, the water surface and energy grade line are usually assumed to coincide at the entrance; the approach velocity head is ignored. The same can be said for the downstream velocity head. That being the case, the approach velocity head and downstream velocity head terms in the above equation would be dropped and the equation would take the form below. Note that this equation has been organized to provide the resulting headwater depth.

$$HW = h_o + H - L \bullet S_o$$

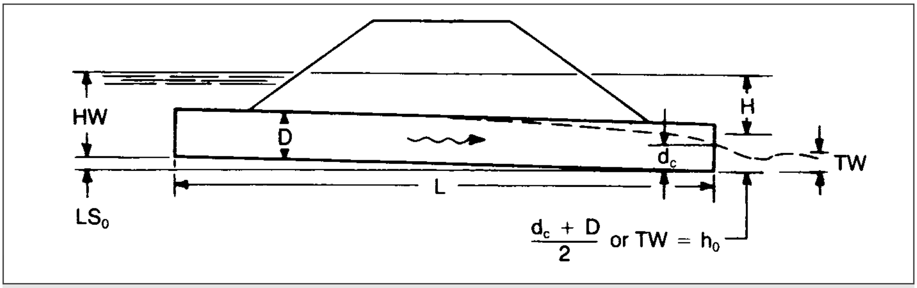
The head, or energy (Figures 4.7 through 4.9) required to pass a given quantity of water through a culvert flowing in outlet control, is made up of a (1) entrance loss, (2) friction loss, and (3) exit loss.



■ **Figure 4.7** Definition of terms in energy balance equation.



■ **Figure 4.8** Terms of the energy balance equation related to a high tailwater condition.



■ **Figure 4.9** Terms of the energy balance equation related to a low tailwater condition.

This energy is expressed in equation form as:

$$H = H_e + H_f + H_o$$

where: H_e = entrance loss, ft
H_f = friction loss, ft
H_o = exit loss, ft

The hydraulic slope, or hydraulic grade line, sometimes called the pressure line, is defined by the elevations to which water would rise in small vertical pipes attached to the culvert wall along its length (see Figure 4.7). For full flow, the energy grade line and hydraulic grade line are parallel over the length of the barrel except in the vicinity of the inlet where the flow contracts and re-expands. The difference between the energy grade line and hydraulic grade line is the velocity head. It turns out that the velocity head is a common variable in the expressions for entrance, friction and exit loss.

The velocity head is expressed by the following equation:

$$H_v = \frac{V^2}{2g}$$

where: H_v = velocity head, ft
V = mean velocity of flow in the barrel, ft/s = Q / A
Q = design discharge, ft^3/s
A = cross sectional area of the culvert, ft^2

The entrance loss depends upon the geometry of the inlet. This loss is expressed as an entrance loss coefficient multiplied by the velocity head, or:

$$H_v = k_e \frac{V^2}{2g}$$

where: k_e = entrance loss coefficient (Table 4.5)

The friction loss is the energy required to overcome the roughness of the culvert barrel and is expressed by the following equation:

$$H_f = \left\{\frac{29n^2L}{R^{1.33}}\right\}\frac{V^2}{2g}$$

where: n = Manning's friction factor (see Tables 4.6 and 4.7)
R = hydraulic radius, ft = A / WP
WP = wetted perimeter, ft

The exit loss depends on the change in velocity at the outlet of the culvert. For a sudden expansion, the exit loss is expressed as:

$$H_o = 1.0\left[\frac{V^2}{2g} - \frac{V_2^2}{2g}\right]$$

As discussed previously, the downstream velocity head is usually neglected, in which case the above equation becomes the equation for the velocity head:

$$H_o = H_v = \frac{V^2}{2g}$$

Substituting in the equation for head we get (for full flow):

$$H_o = \left\{k_e + \frac{29n^2L}{R^{1.33}} + 1\right\}\frac{V^2}{2g}$$

Nomographs have been developed and can be used for solving this equation. Note that these nomographs provide the head, whereas the inlet control nomographs provide the headwater depth. The head is then used to calculate the headwater depth by solving the preceding equation for HW (including the terms of h_o and $L \bullet S_o$).

This equation was developed for the full flow condition, which is as shown in Figure 4.5 A. It is also applicable to the flow condition shown in Figure 4.5 B.

Backwater calculations are required for the partly full flow conditions shown in Figure 4.5 C and D. These calculations begin at the downstream water surface and proceed upstream to the entrance of the culvert and the headwater surface. The downstream water surface is based on either the critical depth or the tailwater depth, whichever is greater (Figure 4.9).

Table 4.6

Values of coefficient of roughness (Manning's *n*) for standard corrugated steel pipes

		2-2/3 x 1/2	Helical Corrugation, Pitch x Rise (in.)										
		Annular	1-1/2 x 1/4		2-2/3 x 1/2								
Flowing	Finish	Corrugation	Diameter (in.)										
		All Dia.	8	10	12	15	18	24	30	36	42	48	≥>= 54
Full	Unpaved	0.024	0.012	0.014	0.011	0.012	0.013	0.015	0.017	0.018	0.019	0.020	0.021
Full	25% paved	0.021						0.014	0.016	0.017	0.018	0.020	0.019
Part Full	Unpaved	0.027			0.012	0.013	0.015	0.017	0.019	0.020	0.021	0.022	0.023
		All	Pipe Arch Span x Rise (in.)										
		Pipe Arches				17 x 13	21 x 15	28 x 20	35 x 24	42 x 29	49 x 33	57 x 38	≥>= 54 x 43
Full	Unpaved	0.026				0.013	0.014	0.016	0.018	0.019	0.020	0.021	0.022
Part Full	Unpaved	0.029				0.018	0.016	0.021	0.023	0.024	0.025	0.025	0.026
		3 x 1	Helical Corrugation, Pitch x Rise (in.)										
		Annular	3 x 1										
		Corrugation	Diameter (in.)										
		All Dia.				36	42	48	54	60	66	72	≥>=78
Full	Unpaved	0.027				0.022	0.022	0.023	0.023	0.024	0.025	0.026	0.027
Full	25% Paved	0.023				0.019	0.019	0.020	0.020	0.021	0.022	0.022	0.023
		5 x 1	Helical Corrugation, Pitch x Rise (in.)										
		Annular	5 x 1										
		Corrugation	Diameter (in.)										
		All Dia.						48	54	60	66	72	≥>=78
Full	Unpaved	0.025						0.022	0.022	0.023	0.024	0.024	0.025
Full	25% Paved	0.022						0.019	0.019	0.020	0.021	0.021	0.022
			All Diameters										
Smooth Interior Pipe (1)			0.012										

Note (1): Includes fully paved, concrete lined. ribbed pipe and double wall pipe.

Table 4.7

Values of coefficient of roughness (Manning's *n*) for structural plate pipe, 6 in. x 2 in. corrugations

Corrungations 6 x 2 in.	Diameters			
	5 ft	7 ft	10 ft	15ft
Plain – unpaved	0.033	0.032	0.030	0.028
25% Paved	0.028	0.027	0.026	0.024

The backwater calculations can be tedious and time consuming. Approximation methods have therefore been developed for the analysis of partly full flow conditions. Backwater calculations have shown that a downstream extension of the full flow hydraulic grade line, for the flow condition shown in Figure 4.5 C, intersects the plane of the culvert outlet cross section at a point half way between the critical depth and the top of the culvert. This is more easily envisioned as shown in Figure 4.9. It is possible to begin the hydraulic grade line at that datum point and extend the straight, full flow hydraulic grade line to the inlet of the culvert. The slope of the hydraulic grade line is the full flow friction slope:

$$S_n = \frac{H_f}{L} = \left\{\frac{29n^2}{R^{1.33}}\right\}\frac{V^2}{2g}$$

If the tailwater elevation exceeds the datum point described above, the tailwater depth is used instead as the downstream starting point for the full flow hydraulic grade line.

The headwater depth is calculated by adding the inlet losses to the elevation of the hydraulic grade line at the inlet.

This method approximation works best when the culvert is flowing full for at least part of its length, as shown in Figure 4.5 C. If the culvert is flowing partly full for its whole length, as shown in Figure 4.5 D, the results become increasingly inaccurate as the flow depth decreases. The results are usually acceptable down to a headwater depth of about three quarters of the structure rise. For lower headwater depths, backwater calculations are required.

The outlet control nomographs can by used with this method of approximation. In this case, the head is added to the datum point elevation to obtain the headwater depth. This method also works best when the culvert is flowing full for part of its length, and the results are not as accurate for a culvert flowing partly full.

Research on Values of *n* for Helically Corrugated Steel Pipe

Tests conducted on helically corrugated steel pipe, both round and pipe arch flowing full and part full, demonstrate a lower coefficient of roughness compared to annularly corrugated steel pipe. The roughness coefficient is a function of the corrugation helix angle (angle subtended between corrugation direction and centerline of the corrugated steel pipe), which determines the helically corrugated pipe diameter. A small helix angle associated with small diameter pipe, correlates to a lower roughness coefficient. Similarly, as the helix angle increases with diameter, the roughness coefficient increases, approaching the value associated with annularly corrugated pipe.

Values for 5 x 1 inch corrugations have been based on tests conducted using 6 x 2 inch and subsequently modified for the shorter pitch. Most published values of the coefficient of roughness, n, are based on experimental work conducted under controlled laboratory conditions using clear or clean water. The test pipe lines are straight with smooth joints. However, design values should take into account the actual construction and service conditions, which can vary greatly for different drainage materials. Also, as noted on preceding pages, culvert or storm drain capacity under inlet control flow conditions is not affected by the roughness of pipe material.

Field Studies on Structural Plate Pipe

Model studies by the U.S. Corps of Engineers, and analyses of the results by the U.S. Federal Highway Administration, have been the basis for friction factors of structural plate pipe for many years. These values ranged from 0.0328 for 5 foot diameter pipe to 0.0302 for 15 foot diameter pipe.

In 1968, the first full-scale measurements were made on a 1500 foot long 14 foot diameter structural plate pipe line in Lake Michigan. These measurements indicated a lower friction factor than those derived from the model studies. As a result, the recommended values of Manning's n for structural plate pipe of 10 foot diameter and larger have been modified as shown in Table 4.7. The values for the smaller diameters remain as they were.

HYDRAULIC COMPUTATIONS

A balanced design approach is considered one in which the approach establishes a minimum opening required to pass, for example, a 10-year flood with no ponding. In this example, the 10-year discharge is established from hydrology data. The pipe size required to carry this flow, with no head at the entrance (HW/D = 1.0), is then determined from nomographs. The designer uses the 10-year discharge to determine the pipe size required for inlet control and for outlet control, and uses whichever is greater. This is typically the minimum required opening size for the culvert.

Inlet Control

The headwater (HW) for a given pipe flowing under inlet control can be determined from Figures 4.10 through 4.17. Round pipes, pipe arches, and arches are included, as indicated.

These figures are first used to determine the pipe size required so there is no head at the entrance under a 10-year flood condition. Once a pipe size is chosen, the designer also checks that pipe to determine whether outlet control will govern (as described below), and makes pipe size adjustments accordingly.

The designer uses the selected pipe size to determine the headwater for specific entrance conditions for the 100-year flood discharge under inlet control. If this amount of headwater is acceptable, the chosen size is satisfactory for the full 100-year design discharge under inlet control. If the resulting headwater is too high, a larger size must be selected based on the maximum permissible headwater.

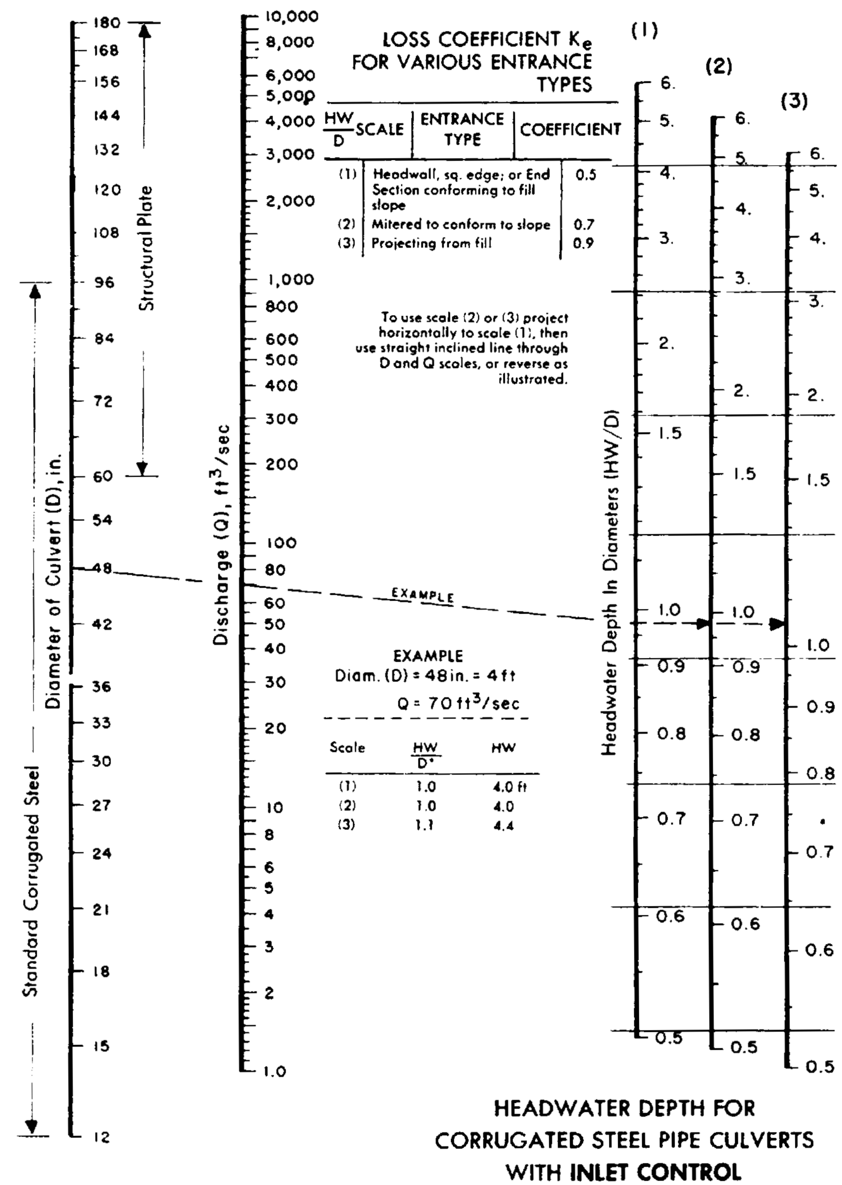
The values from the nomographs give the headwater in terms of a number of pipe rises (HW/D). The following equation is then used to calculate the headwater depth:

$$HW_i = \frac{HW}{D} \bullet D$$

where: HW_i = headwater depth under inlet control, ft

$\frac{HW}{D}$ = headwater depth in number of pipe rises, from nomograph, ft/ft

D = diameter of pipe, or rise of arch or pipe arch, ft



■ **Figure 4.10** Headwater depth for round corrugated steel pipes and structural plate corrugated steel pipes under inlet control.

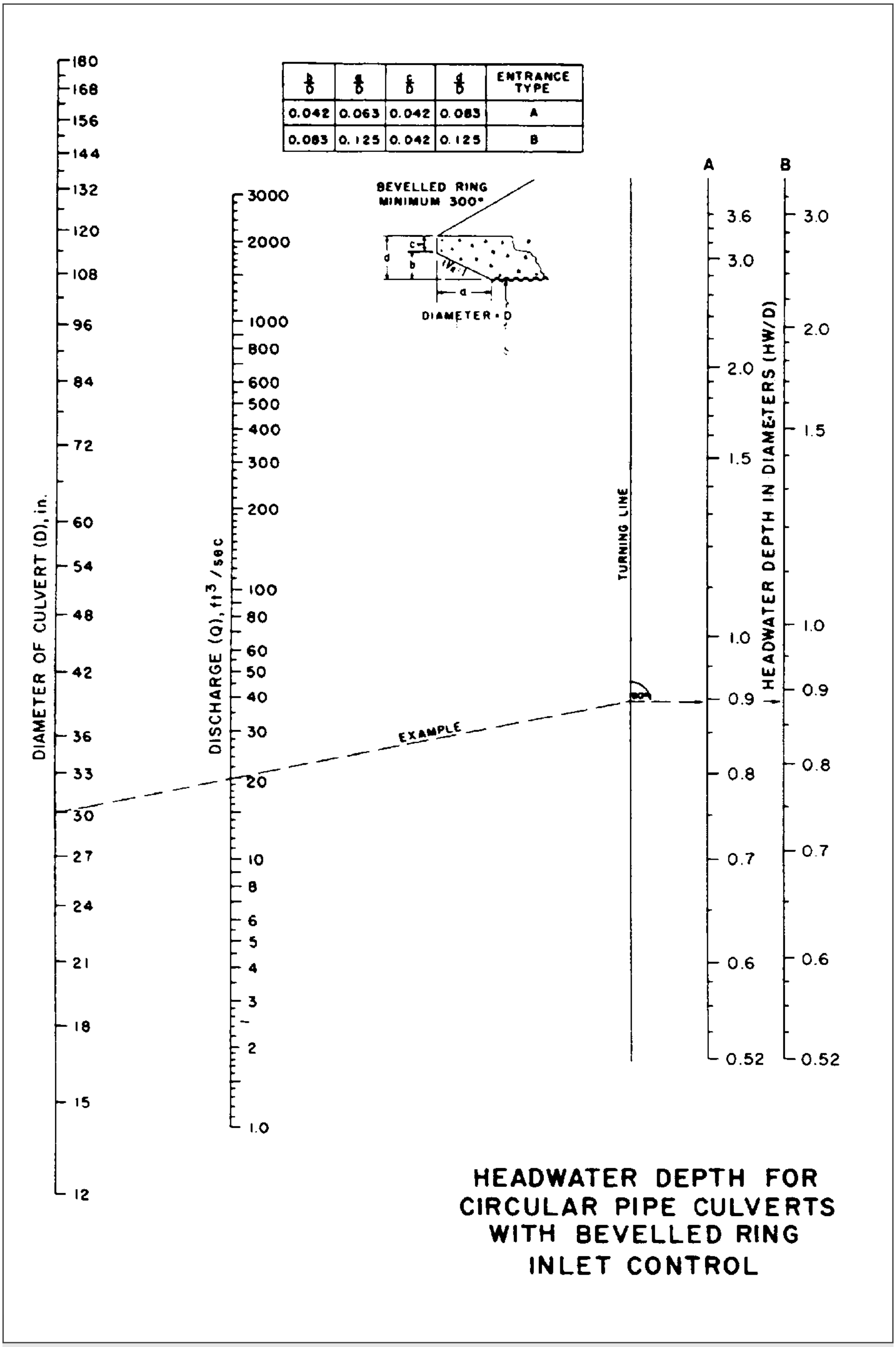
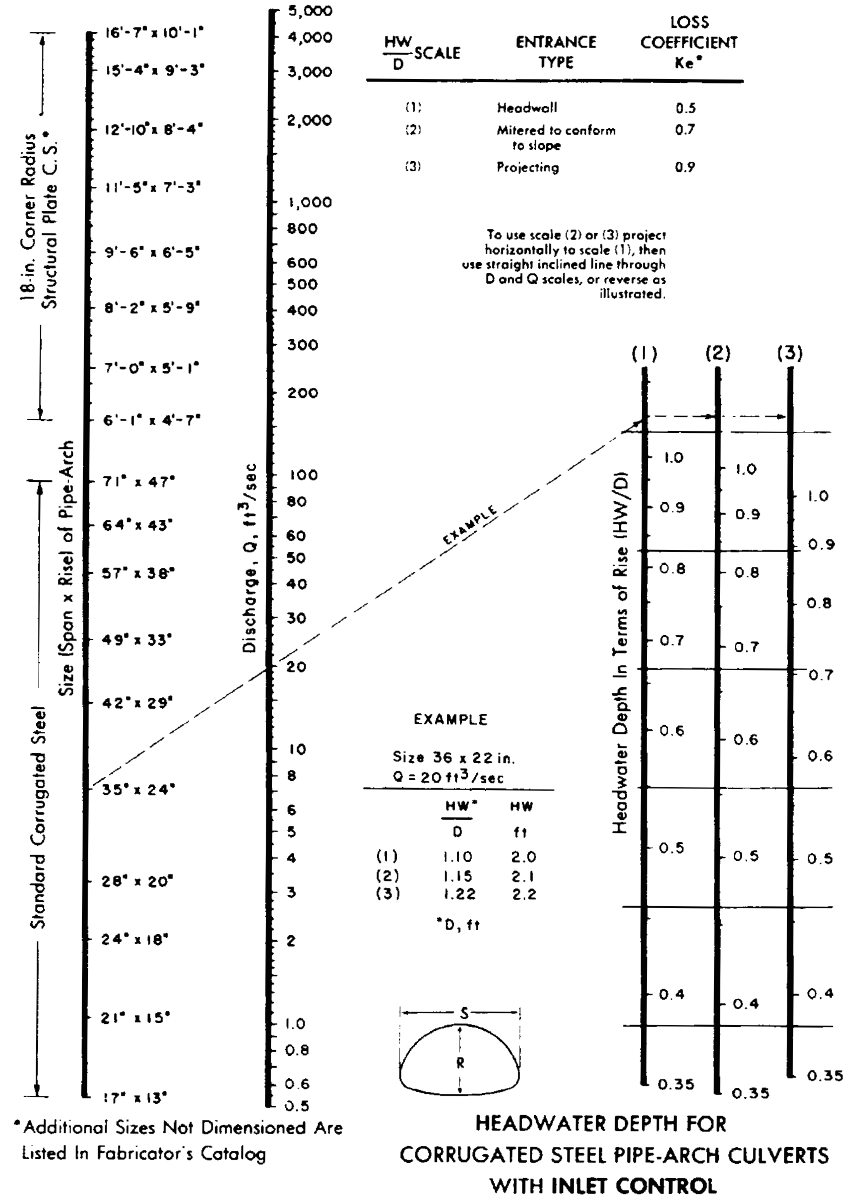
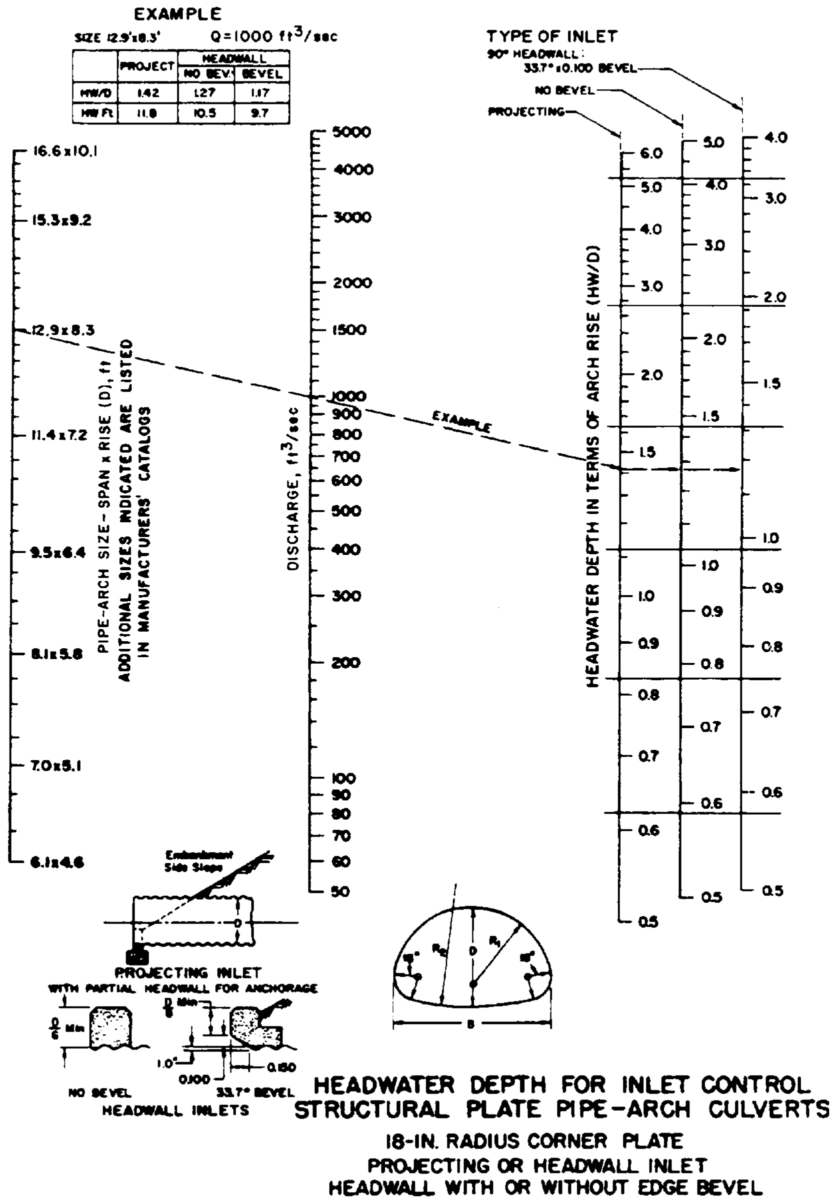


Figure 4.11 Headwater depth for round corrugated steel pipes, with beveled ring headwall, under inlet control.

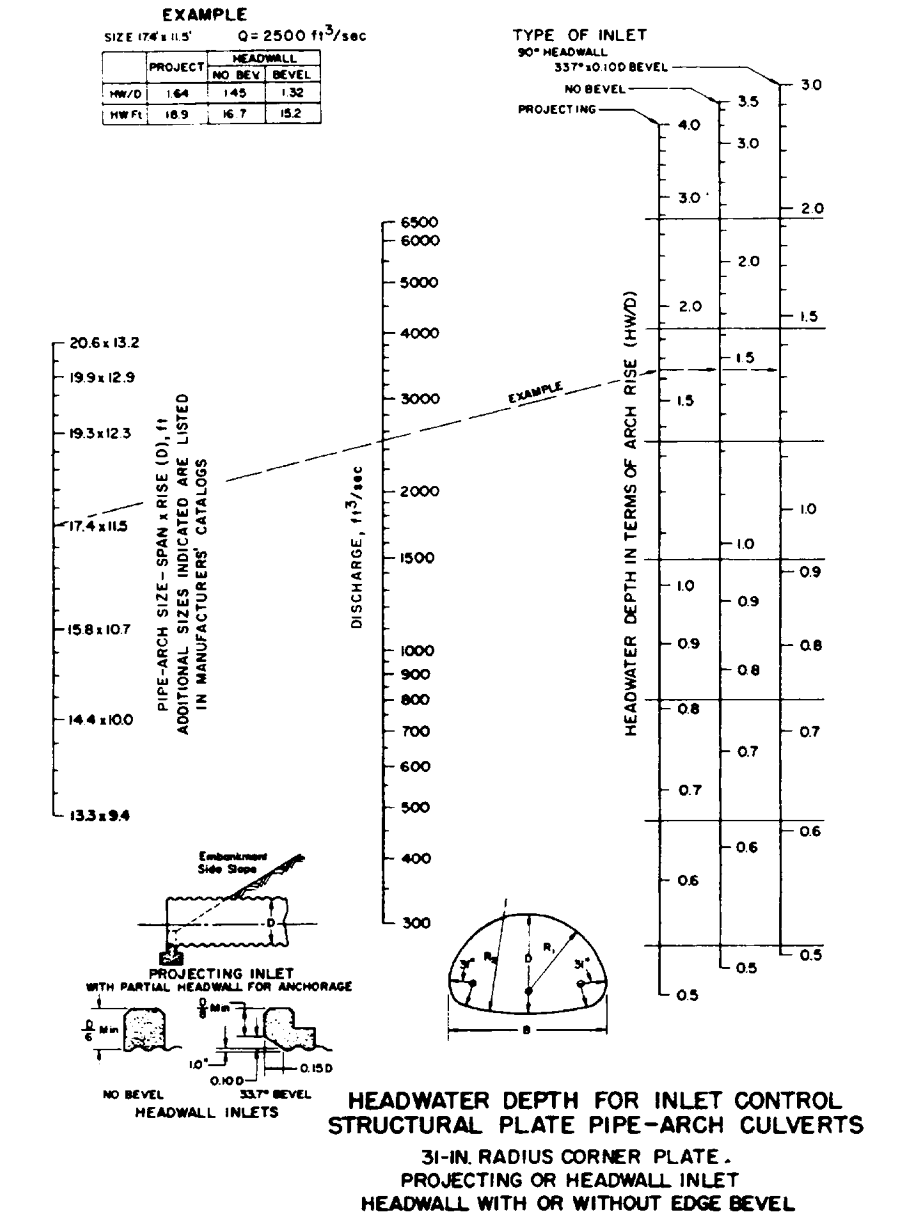


■ **Figure 4.12** Headwater depth for corrugated steel and structural plate corrugated steel pipe arches under inlet control.

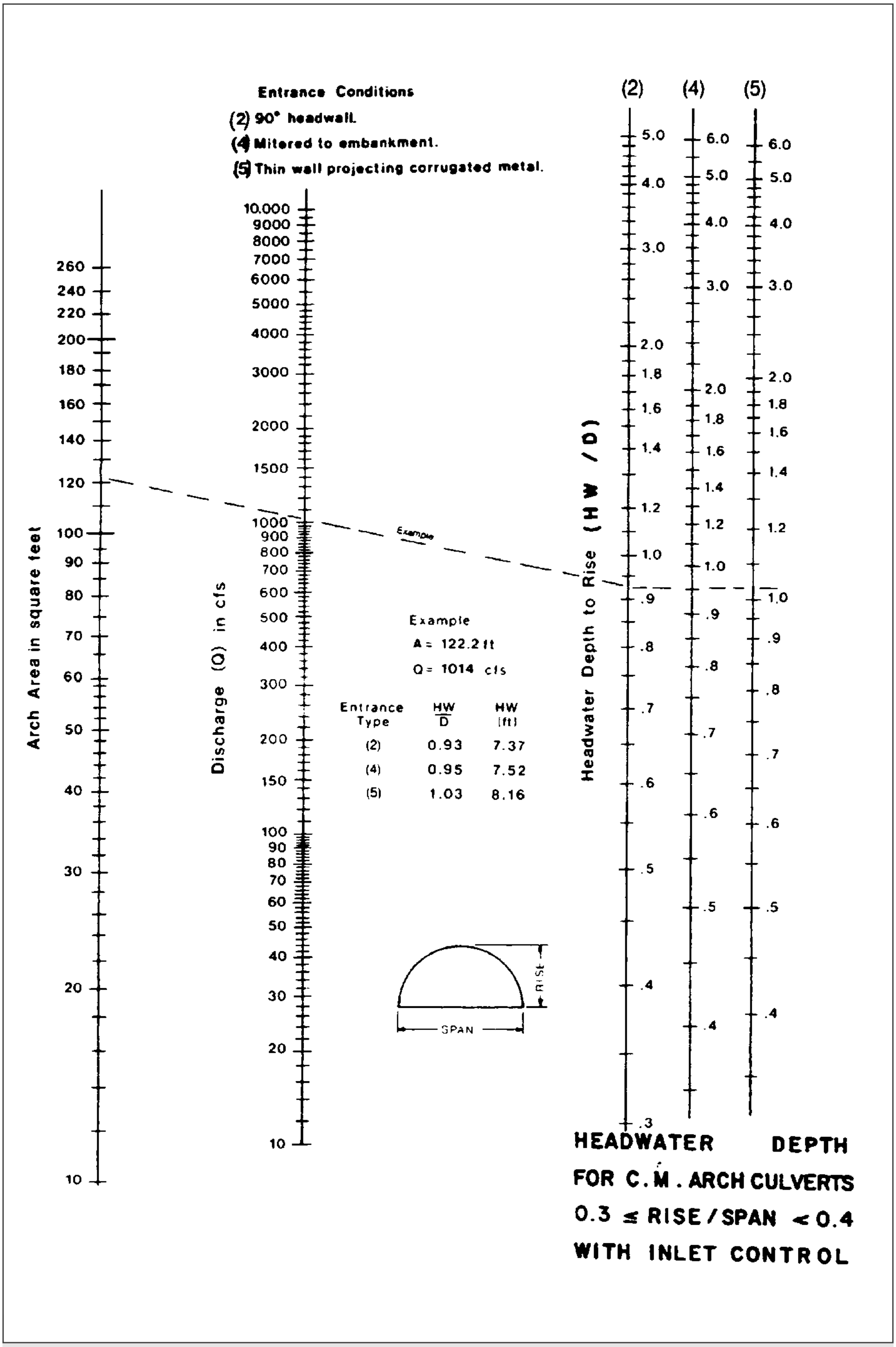


	PROJECT	HEADWALL NO BEV.	HEADWALL BEVEL
HW/D	1.42	1.27	1.17
HW Ft	11.8	10.5	9.7

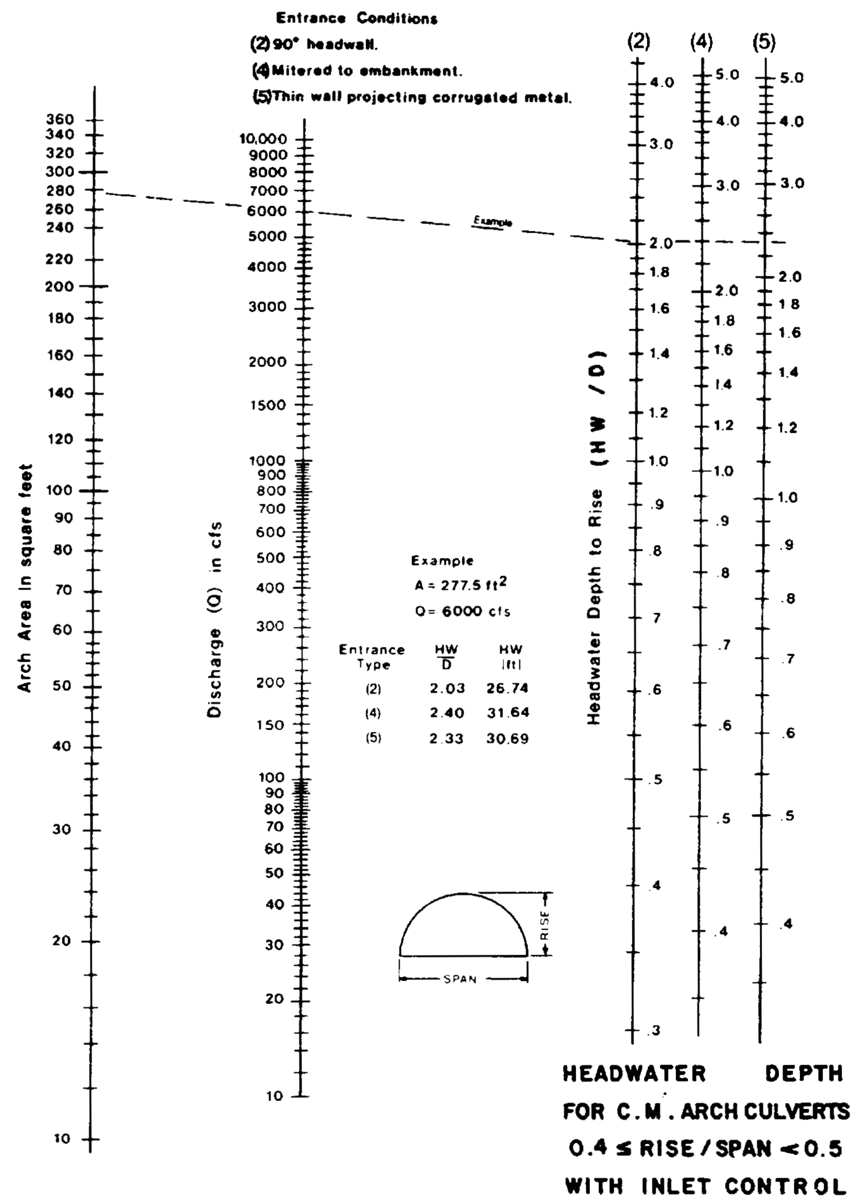
■ **Figure 4.13** Headwater depth for structural plate corrugated steel pipe arches, with 18-in. radius corner plate, under inlet control.



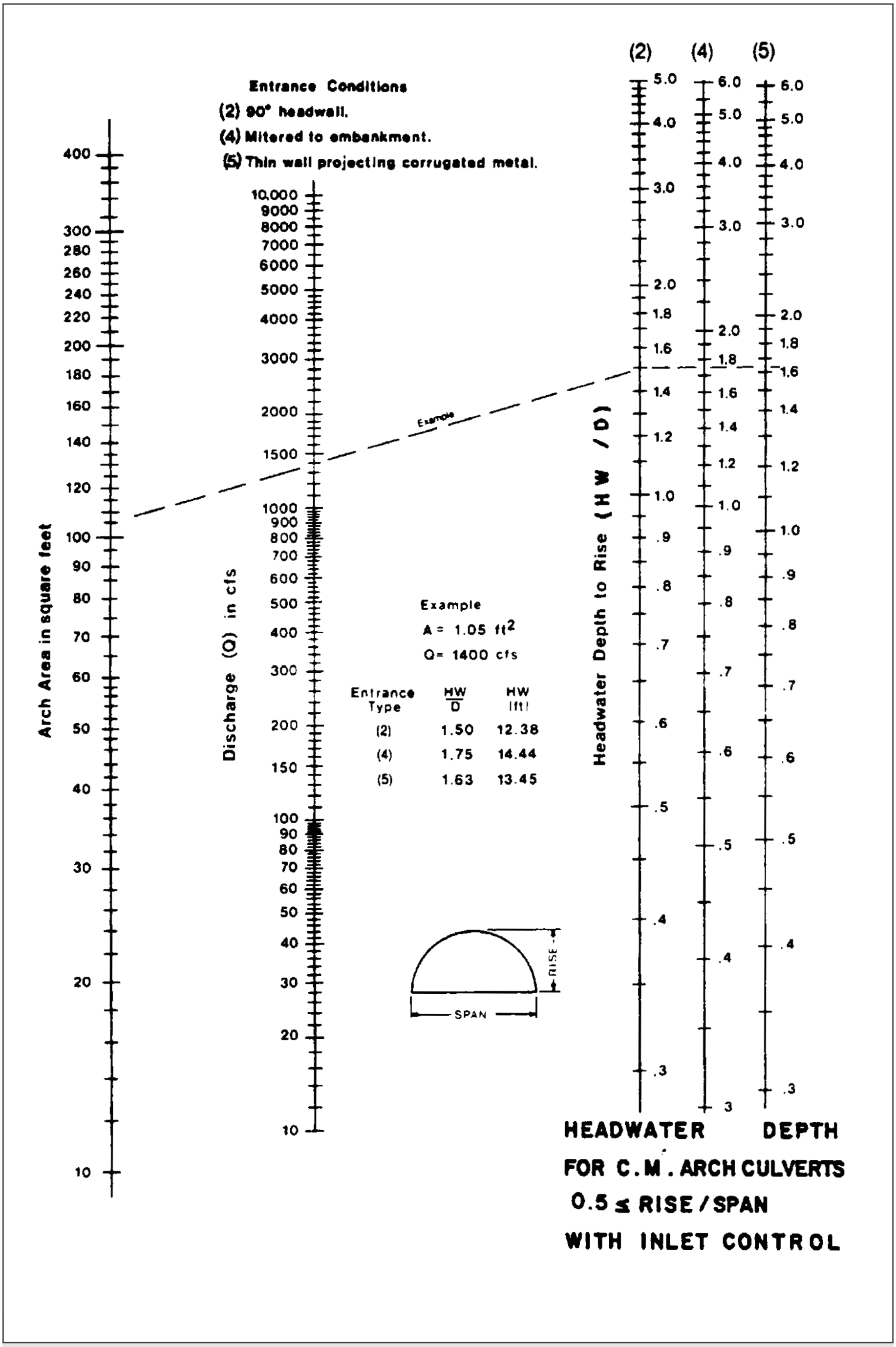
■ **Figure 4.14** Headwater depth for structural plate corrugated steel pipe arches, with 31-in. radius corner plate, under inlet control.



■ **Figure 4.15** Headwater depth for structural plate corrugated steel arches, with 0.3 <= rise/span < 0.4, under inlet control.



■ **Figure 4.16** Headwater depth for structural plate corrugated steel arches, with 0.4 <= rise/span < 0.5, under inlet control.



■ **Figure 4.17** Headwater depth for structural plate corrugated steel arches, with 0.5 <= rise/span, under inlet control.

Outlet Control

Figures 4.18 through 4.27 are used, with the pipe size selected for inlet control, to determine the head loss, H. The head loss is then used in the following equation to determine the headwater depth under outlet control. If the depth computed for outlet control is greater than the depth determined for inlet control, then outlet conditions govern the flow conditions of the culvert and the higher headwater depth applies.

$$HW_o = h_o + H - L \bullet S_o$$

where: HW_o = headwater depth under outlet control, ft
h_o = outlet datum, ft; the greater of the tailwater depth, TW, or $\frac{(d_c + D)}{2}$
H = head, from nomograph, ft
L = length of culvert barrel, ft
S_o = slope of culvert barrel, ft/ft
TW = depth of flow in channel at culvert outlet, ft
d_c = critical depth, from Figures 4.28 through 4.31, ft
D = diameter of pipe, or rise of arch or pipe arch, ft

Wall roughness factors (Manning's *n*), on which the nomographs are based, are stated on each figure. In order to use the nomographs for other values of n, an adjusted value for length, L', is calculated using the equation below. This value is then used on the length scale of the nomograph, rather than the actual culvert length.

$$L' = L \bullet \left(\frac{n'}{n}\right)^2$$

where L′ = adjusted length for use in nomographs, ft
L = actual length, ft
n′ = actual value of Manning's n
n = value of Manning's n on which nomograph is based

Values of Manning's n for standard corrugated steel pipe, which were listed in Table 4.6, are shown for convenience in Table 4.8, together with the corresponding length adjustment factors, $\left(\frac{n'}{n}\right)^2$.

Table 4.8

Length adjustment factors for corrugated steel pipes

Pipe Diameter, in.	Roughness Factor n′ for Helical Corr.	Length Adjustment Factor $\left(\frac{n'}{n}\right)^2$
12	0.011	0.21
24	0.016	0.44
36	0.019	0.61
48	0.020	0.70

Values of Manning's n for structural plate corrugated steel pipe, which were determined in the 1968 full-scale field measurements and were listed in Table 4.7, are shown for convenience in Table 4.9, together with the corresponding length adjustment factors, $\left(\frac{n'}{n}\right)^2$.

Table 4.9

Length adjustment factors for 6 x 2 in. corrugation structural plate pipe

Pipe Diameter, ft	Roughness Factor		Length Adjustment Factor $\left(\frac{n'}{n}\right)^2$
	Curves Based on n =	Actual n' =	
5	0.0328	0.033	1.0
7	0.0320	0.032	1.0
10	0.0311	0.030	0.93
15	0.0302	0.028	0.86
Pipe Arch Size ft	**Roughness Factor**		**Length Adjustment Factor $\left(\frac{n'}{n}\right)^2$**
	Curves Based on n	**Actual n'**	
6.1 x 4.6	0.0327	0.0327	1.0
8.1 x 5.8	0.0321	0.032	1.0
11.4 x 7.2	0.0315	0.030	0.907
16.6 x 10.1	0.0306	0.028	0.837

An appropriate entrance loss curve is used based on the desired entrance condition. Typical values of the entrance loss coefficient, k_e, for a variety of inlet configurations, are listed in Table 4.5.

If outlet control governs the capacity of the culvert and the headwater exceeds the maximum allowable value, a larger size pipe can be selected so that an acceptable headwater depth results. In such a case, corrugated steel structures with lower roughness coefficients should be considered. See Table 4.6 for alternatives. A smaller size of paved pipe, a helical pipe or a ribbed pipe may be satisfactory.

Entrance conditions should also be considered. It may be economical to use a more efficient entrance than originally considered if a pipe size difference results. This can be easily investigated by checking the pipe capacity using other entrance loss coefficient curves.

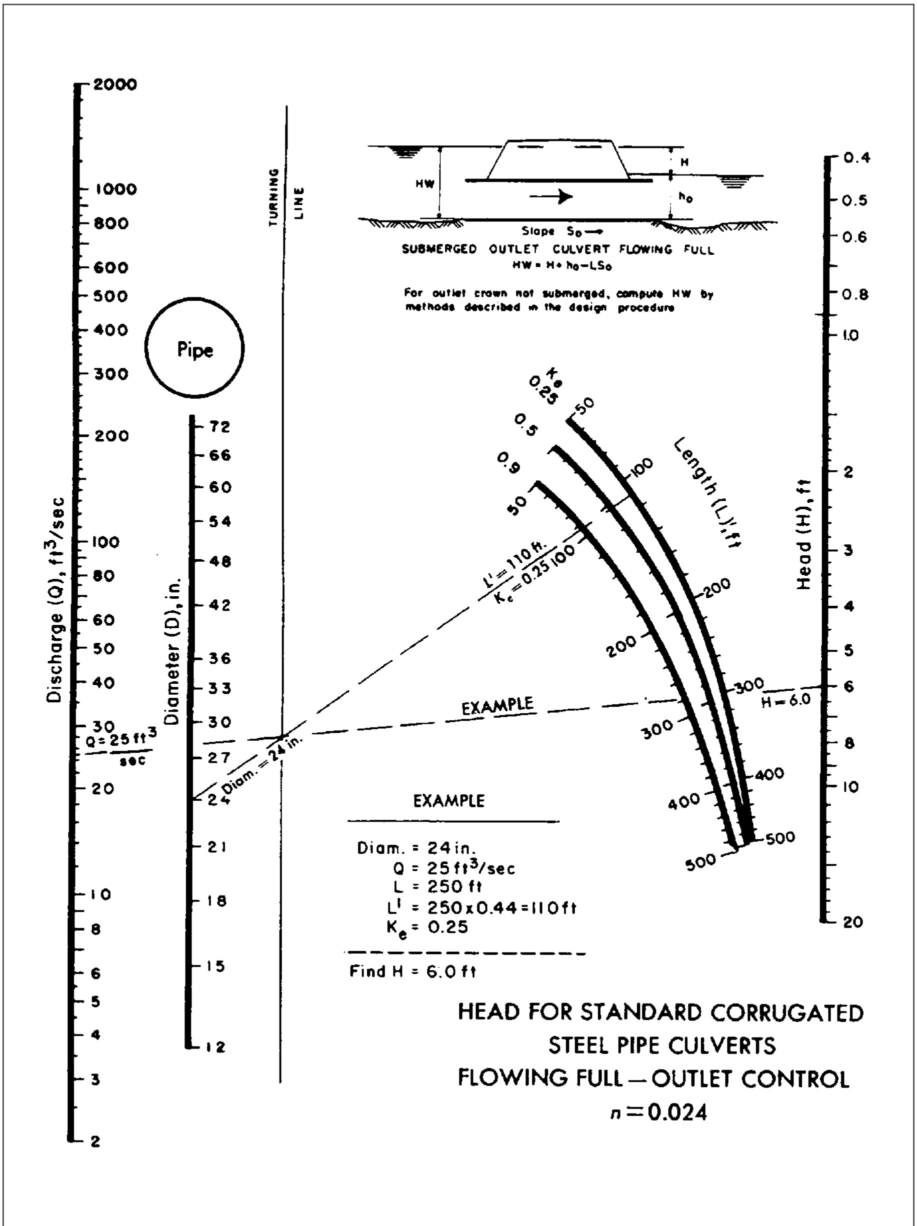


Figure 4.18 Head for round corrugated steel pipes flowing full under outlet control.

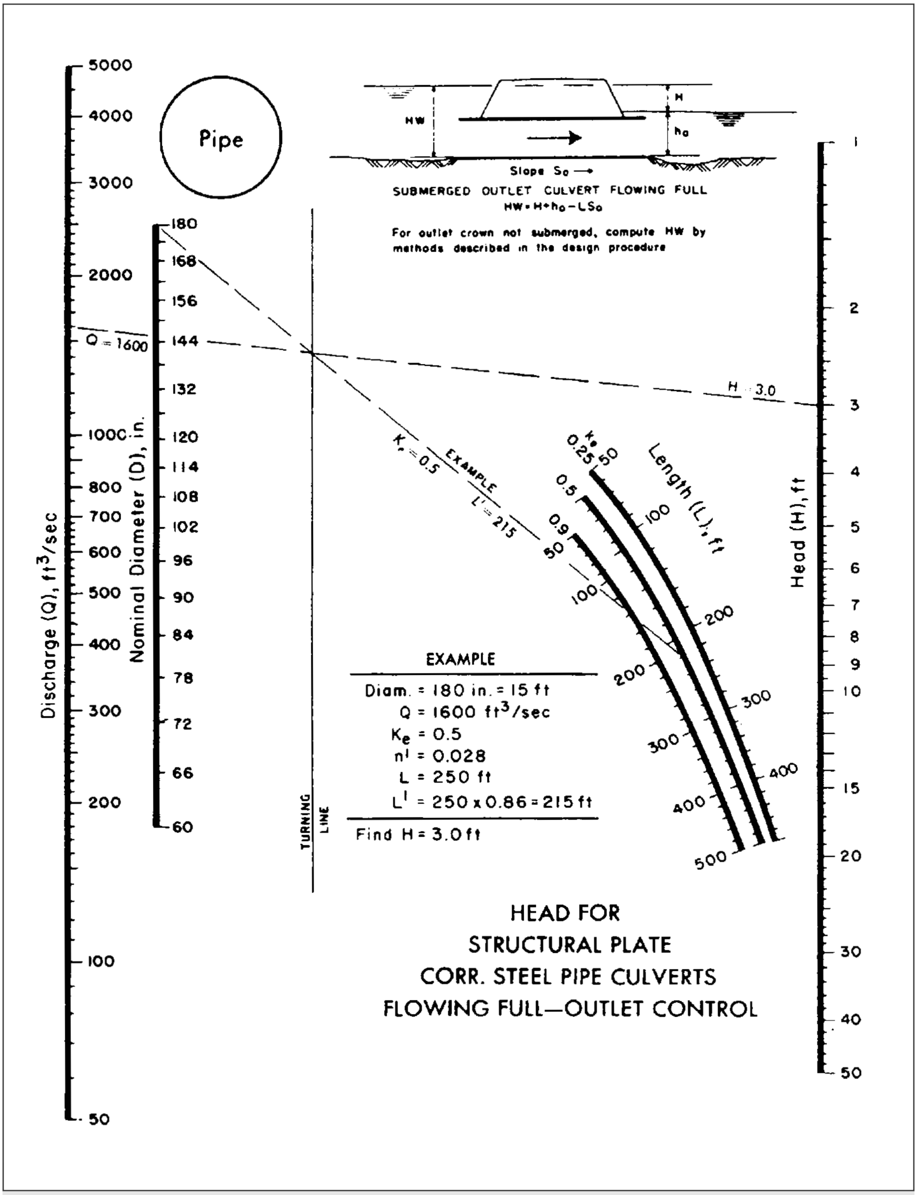


Figure 4.19 Head for round structural plate corrugated steel pipes flowing full under outlet control.

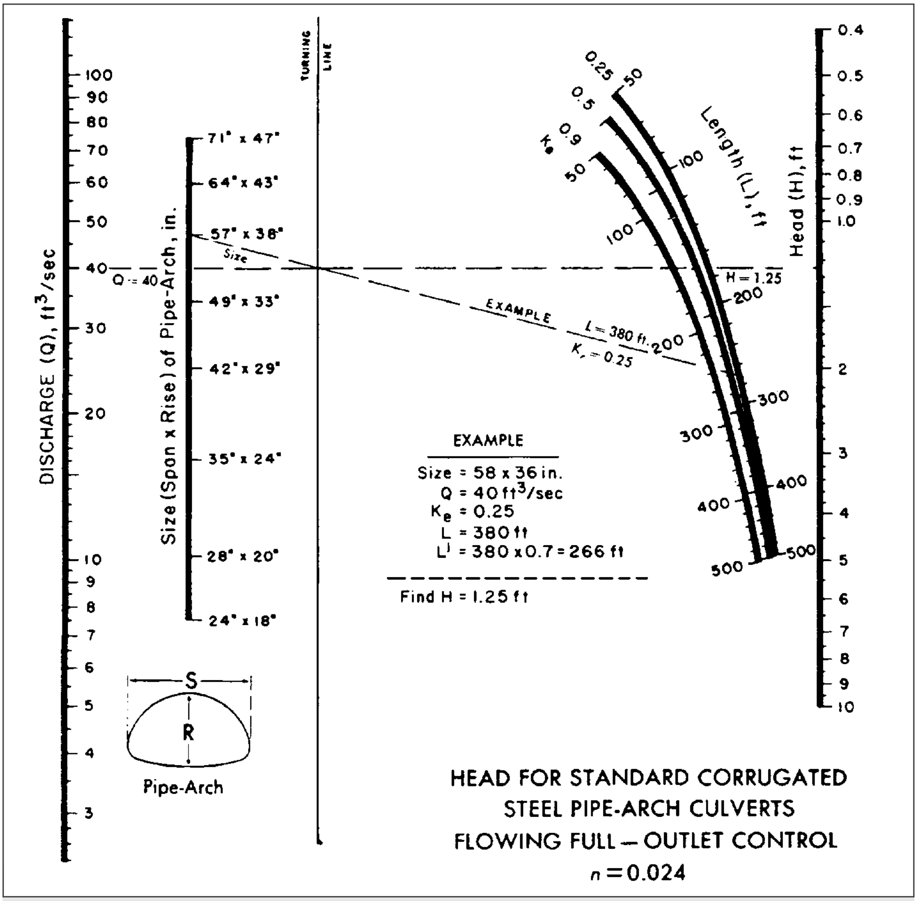
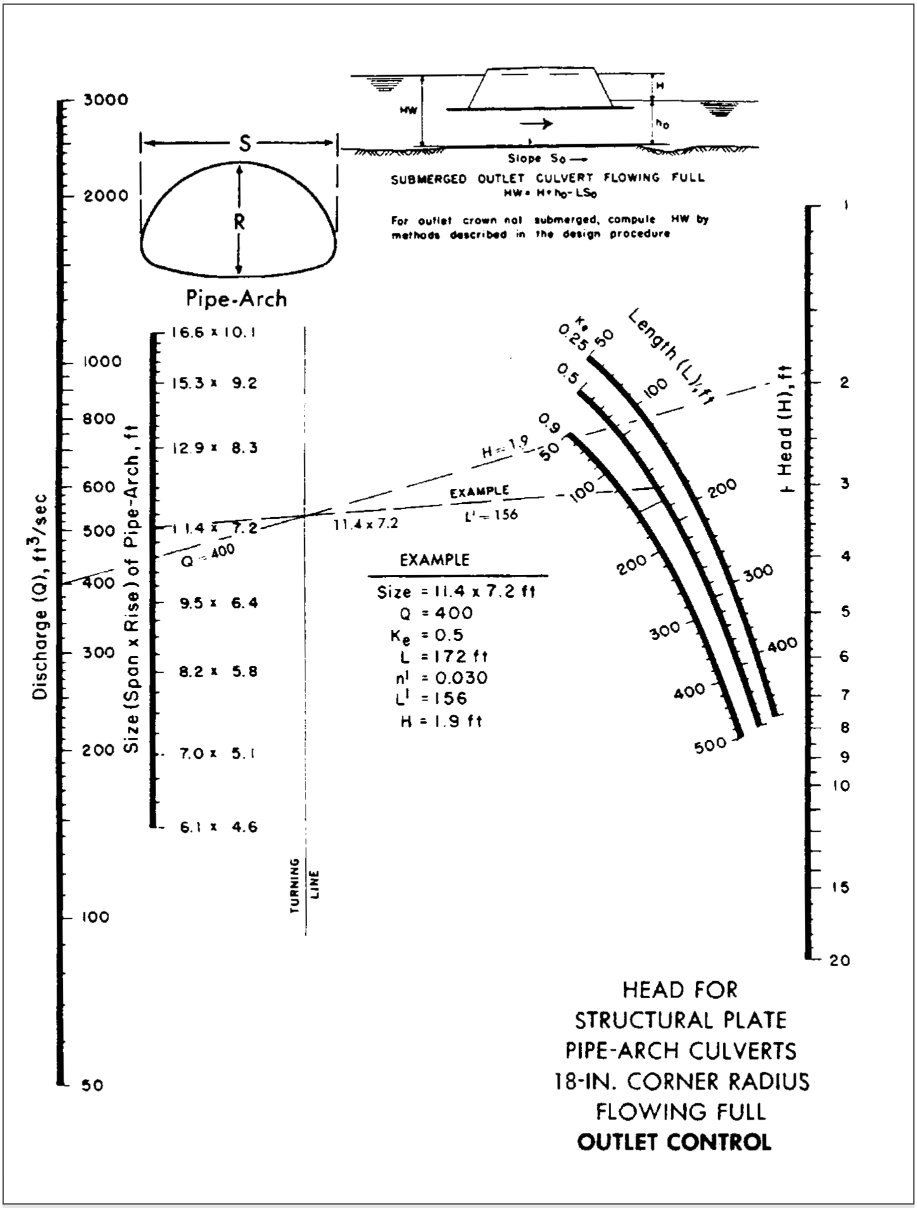
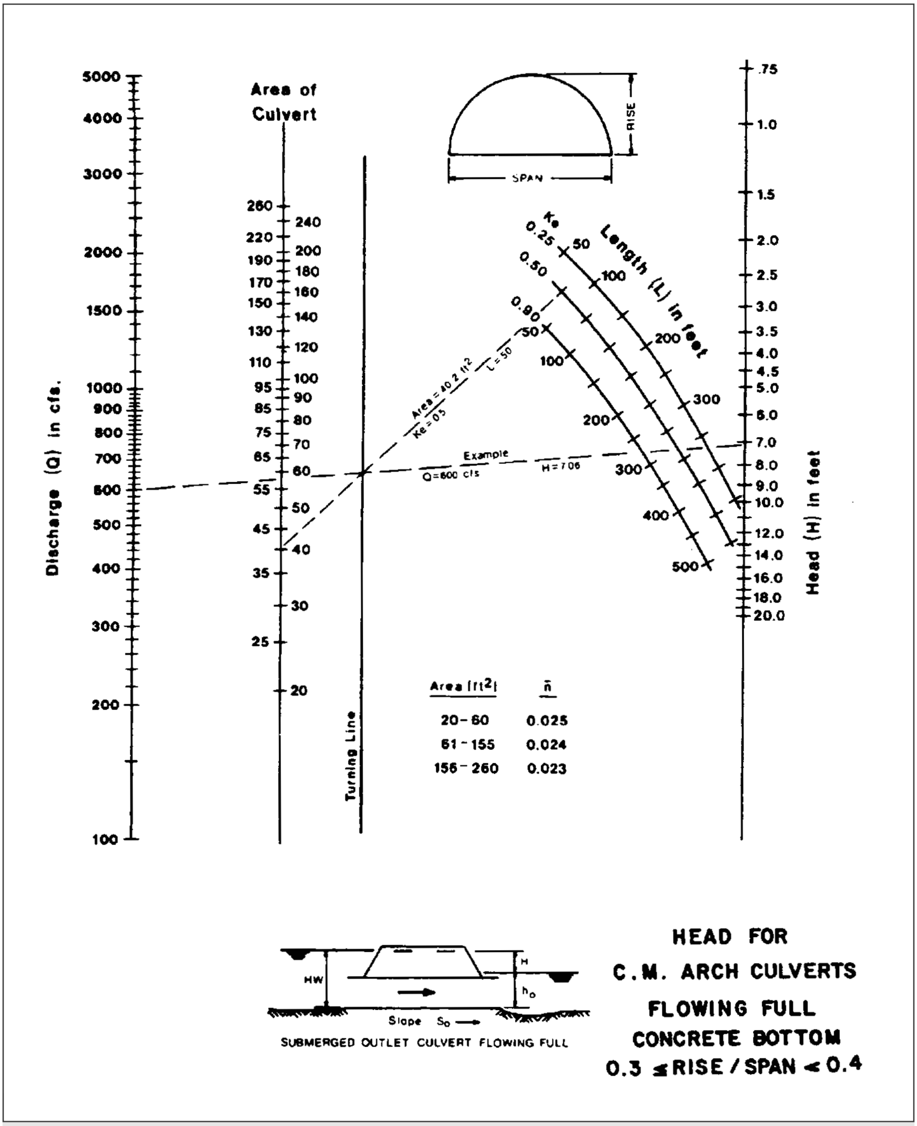


Figure 4.20 Head for corrugated steel pipe arches flowing full under outlet control.

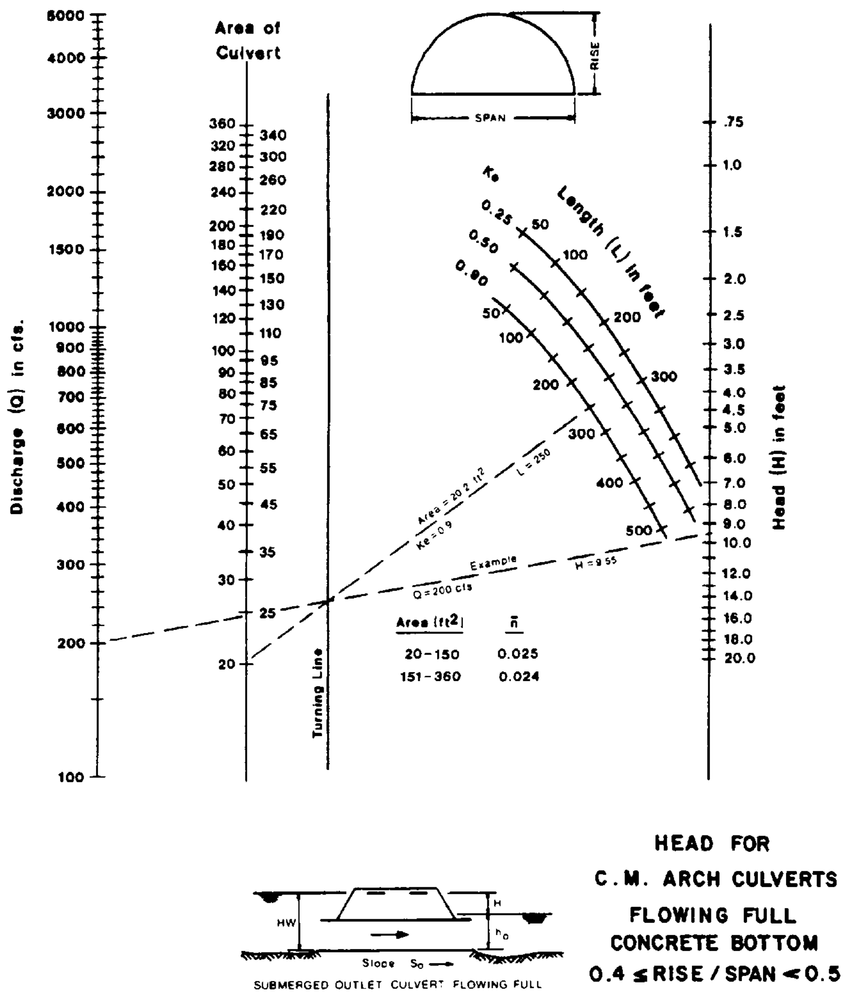
Structural plate pipe arch for an irrigation ditch crossing.



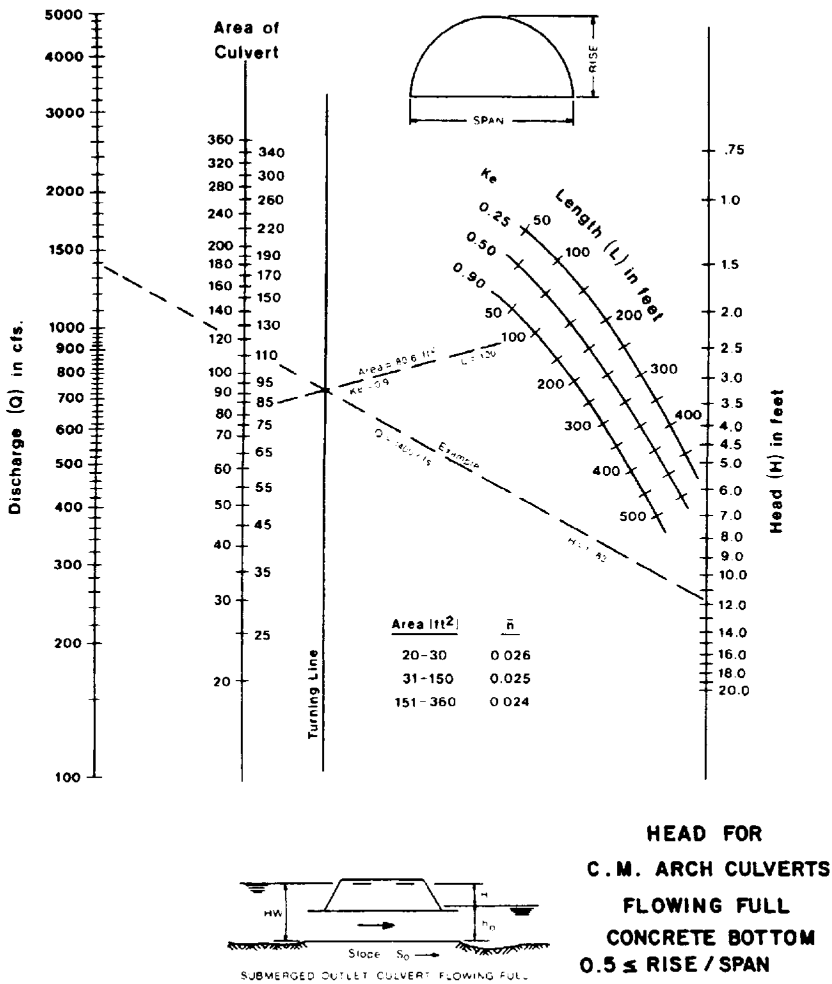
■ **Figure 4.21** Head for structural plate corrugated steel pipe arches with 18-in. corner radius, with submerged outlet and flowing full under outlet control. For 31-in. corner radius structures, use structure sizes on the size scale with equivalent end areas.



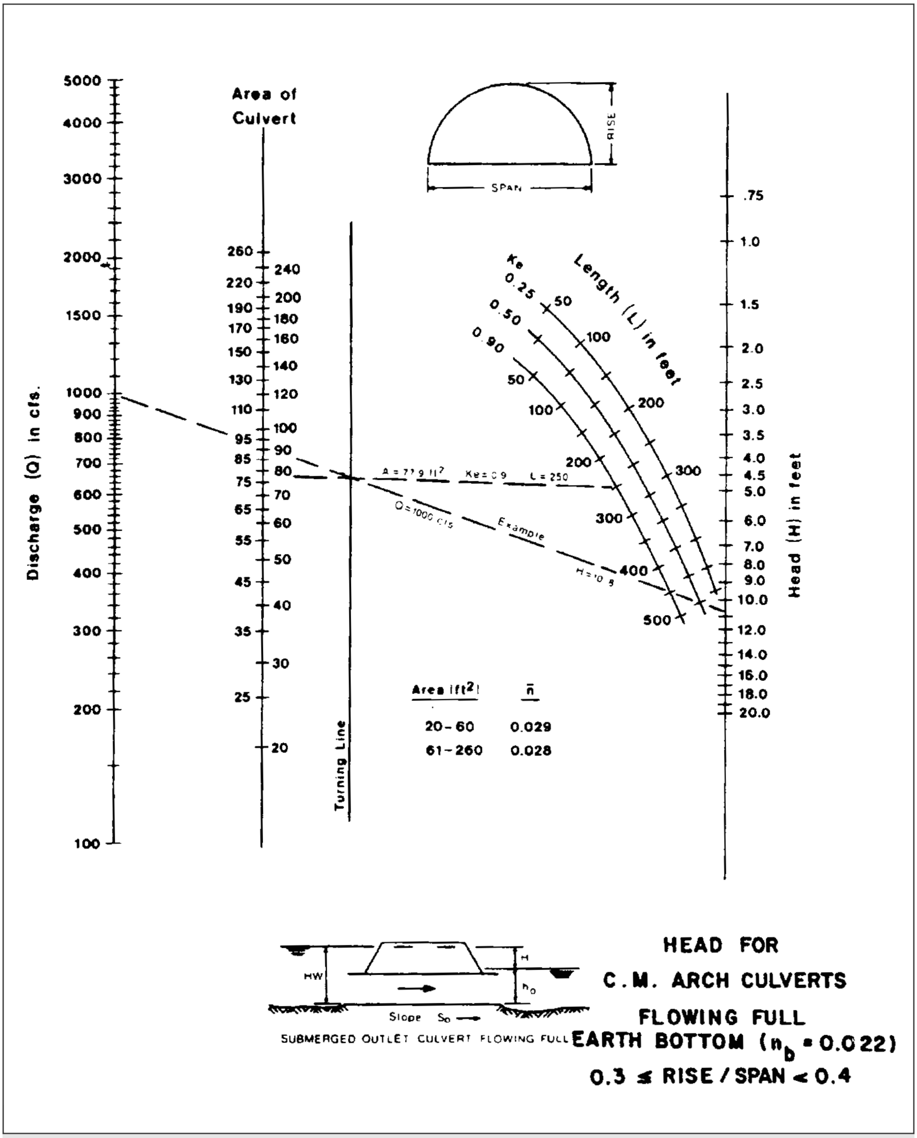
■ **Figure 4.22** Head for structural plate corrugated steel arches, with concrete bottom and 0.3 <= rise/span < 0.4, flowing full under outlet control.



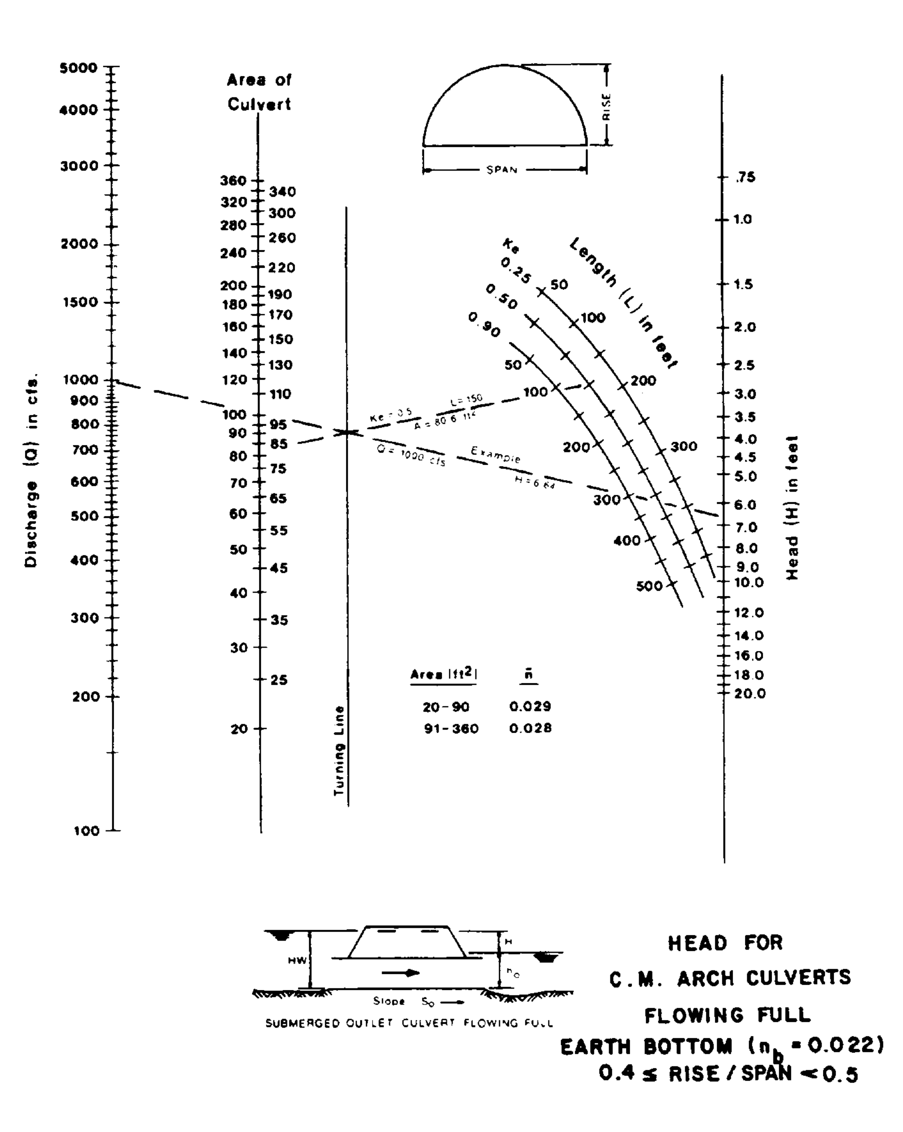
■ **Figure 4.23** Head for structural plate corrugated steel arches, with concrete bottom and 0.4 <= rise/span < 0.5, flowing full under outlet control.



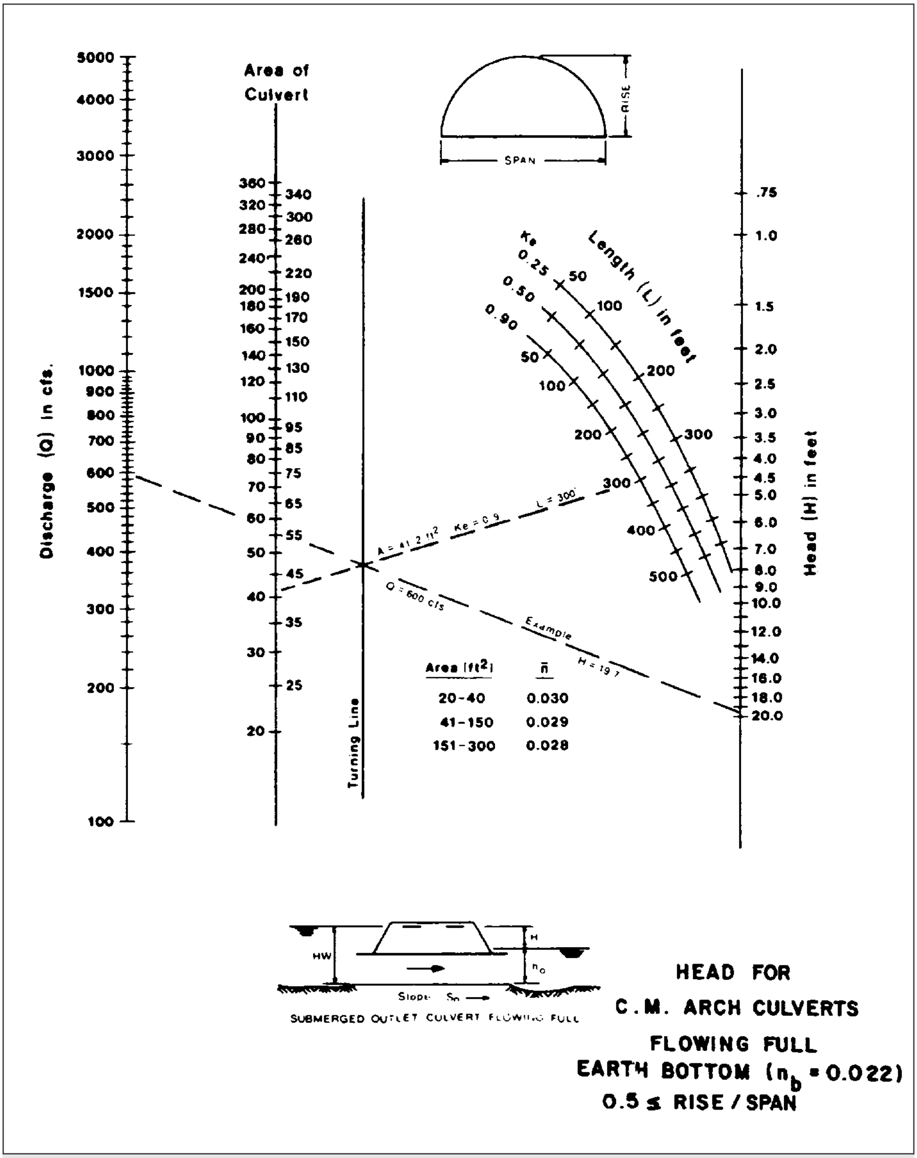
■ **Figure 4.24** Head for structural plate corrugated steel arches, with concrete bottom and 0.5 <= rise/span, flowing full under outlet control.



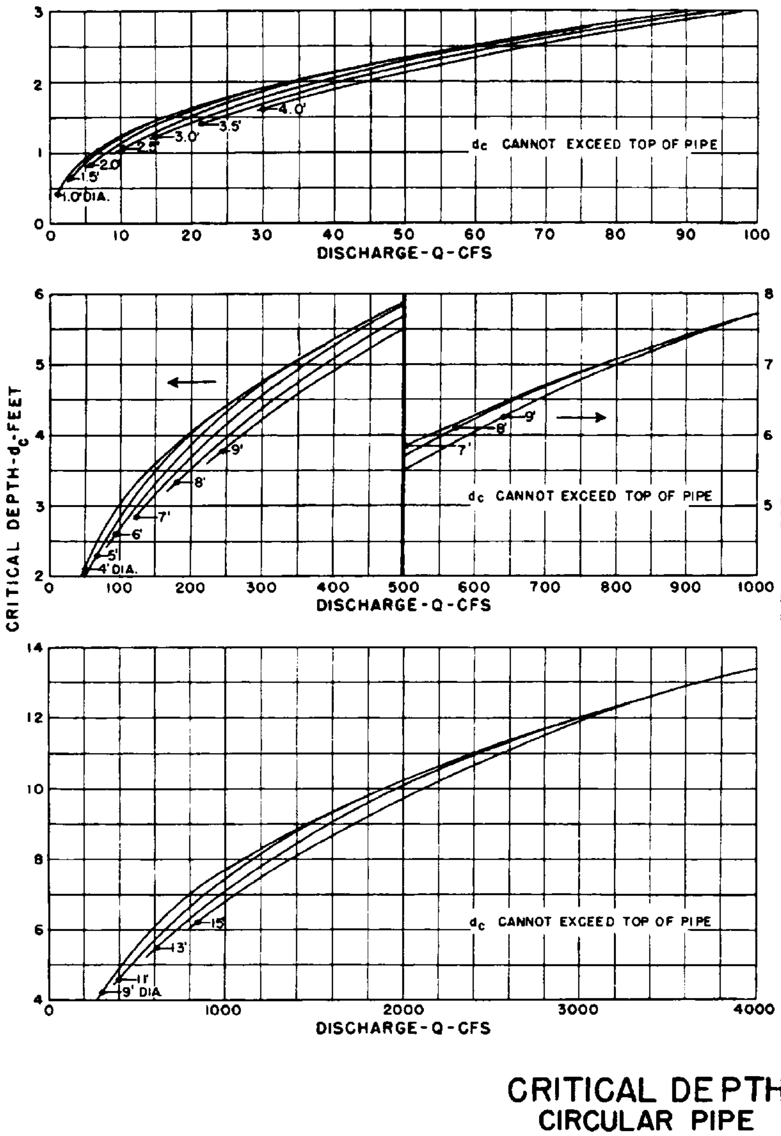
■ **Figure 4.25** Head for structural plate corrugated steel arches, with earth bottom and 0.3 <= rise/span < 0.4, flowing full under outlet control.



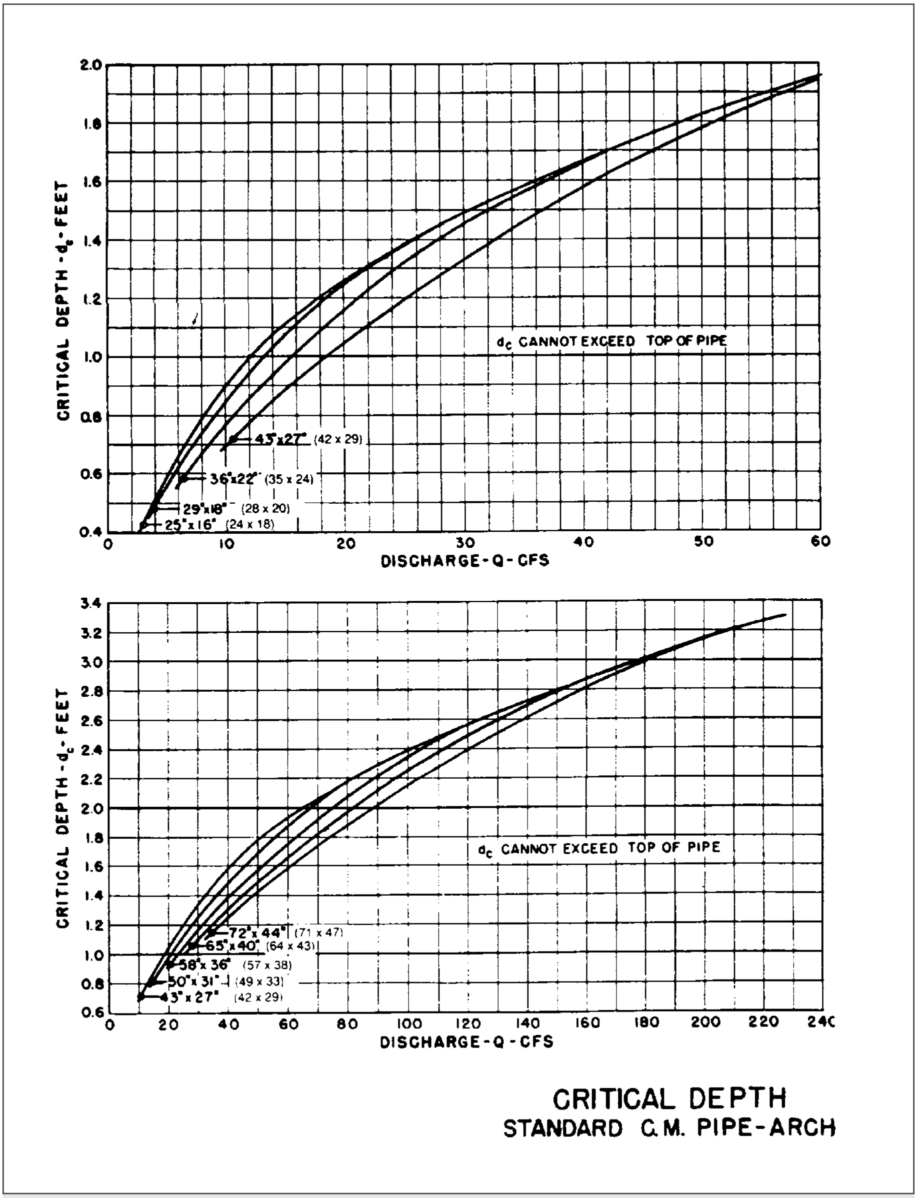
■ **Figure 4.26** Head for structural plate corrugated steel arches, with earth bottom and 0.4 <= rise/span < 0.5, flowing full under outlet control.



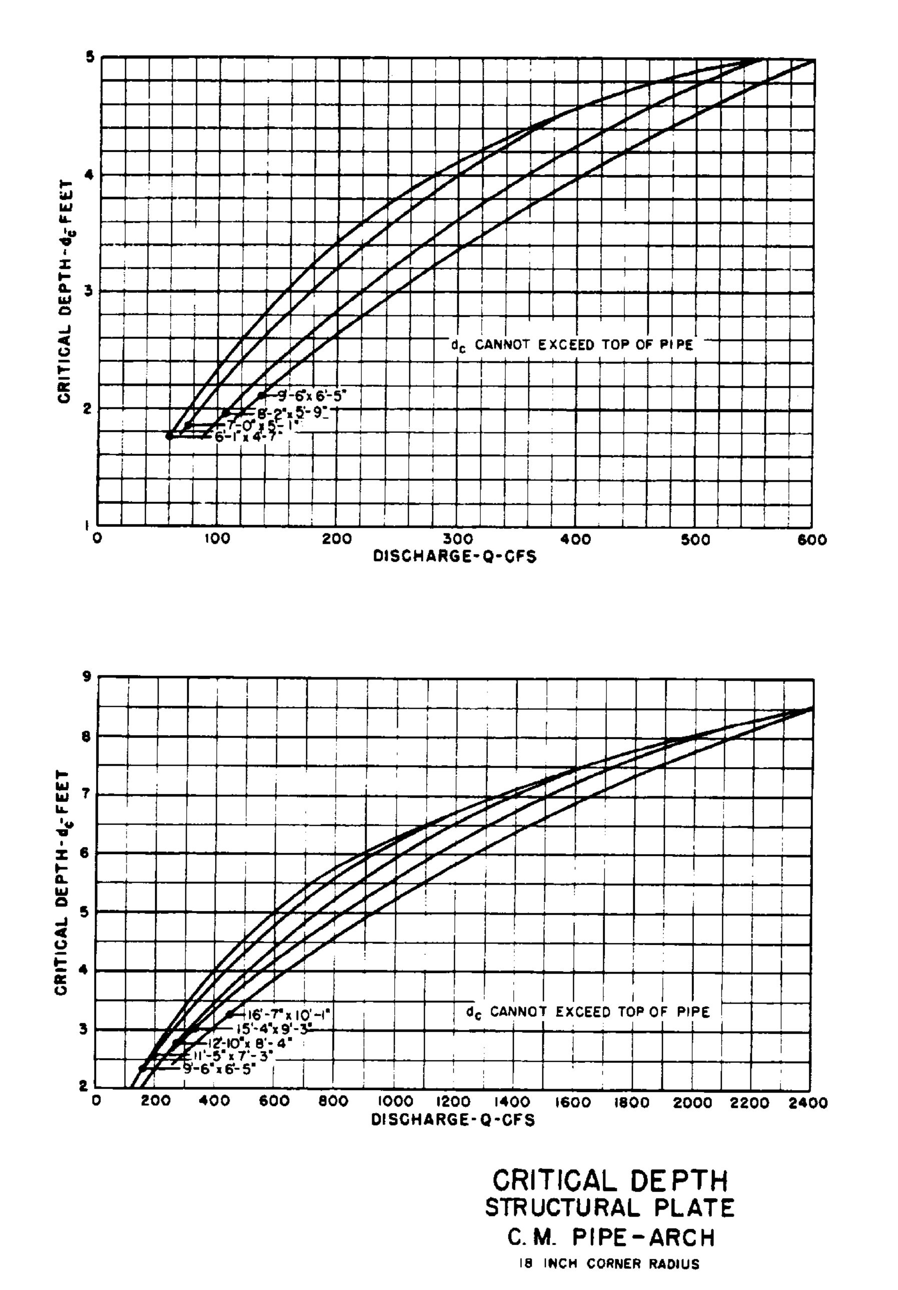
■ **Figure 4.27** Head for structural plate corrugated steel arches, with earth bottom and 0.5 <= rise/span, flowing full under outlet control.



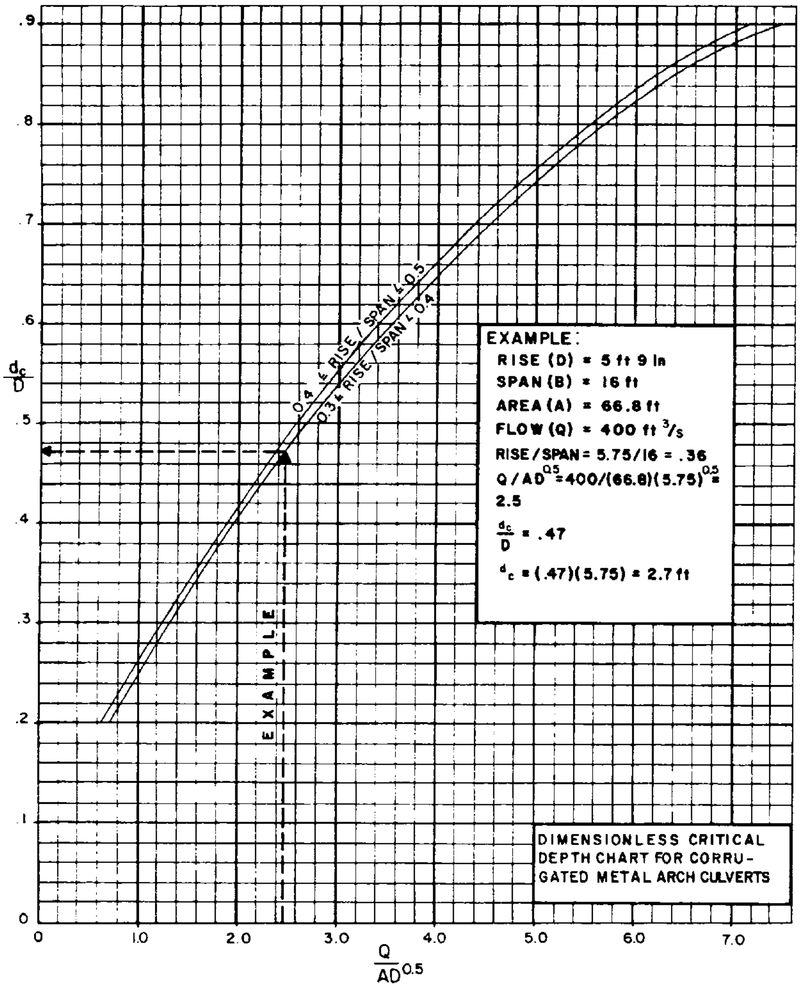
■ **Figure 4.28** Critical depth for round corrugated steel and structural plate corrugated steel pipes.



■ **Figure 4.29** Critical depth for corrugated steel pipe arches.



■ **Figure 4.30** Critical depth for structural plate corrugated steel pipe arches.



■ **Figure 4.31** Critical depth for structural plate corrugated steel arches.

Improved Inlets

Culvert capacity may be increased through the use of special inlet designs. The U.S. Federal Highway Administration (FHWA) has developed design methods for these types of structures. While these designs increase the flow, their use has been limited as a result of their cost and the level of knowledge of designers.

Hydraulic Nomographs

The inlet and outlet control design nomographs which appear in this design manual (Figures 4.10 through 4.27) were reproduced from nomographs developed and published by the FHWA. A certain degree of error is introduced into the design process due to the fact that the construction of nomographs involves graphical fitting techniques resulting in scales which do not exactly match equation results. All of the nomographs used in this design manual have a precision which is better that ±10 percent of the equation value in terms of headwater depth (inlet control) or head loss (outlet control). This degree of precision is usually acceptable, especially when considering the degree of accuracy of the hydrologic data. If a structure size is not shown on a particular nomograph, accuracy is not drastically affected when a user interpolates between known points.

Partly Full Flow

The pipe capacities derived from the above work are for pipes flowing full. Tables 4.10 through 4.14 provide full flow end areas and hydraulic radii for a variety of pipe shapes and sizes. Figures 4.32 through 4.34 provide the means to determine hydraulic section parameters for pipes flowing partly full.

The pipe arch shape is used when a low cover situation requires a pipe with less rise or when a larger flow area is desired for a given flow depth. Figure 4.35 shows a comparison, for an equivalent periphery round and pipe arch, of flow areas for a number of flow depths.

Table 4.10

Area and hydraulic radius for round pipe flowing full

Diameter in.	Area ft^2	Hydraulic Radius, ft	Diameter in.	Area ft^2	Hydraulic Radius, ft
12	0.8	0.250	156	132.7	3.250
15	1.2	0.312	162	143.1	3.375
18	1.8	0.375	168	153.9	3.500
21	2.4	0.437	174	165.1	3.625
24	3.1	0.500	180	176.7	3.750
30	4.9	0.625	186	188.7	3.875
36	7.1	0.750	192	201.1	4.000
42	9.6	0.875	198	213.8	4.125
48	12.6	1.000	204	227.0	4.250
54	15.9	1.125	210	240.5	4.375
60	19.6	1.250	216	254.5	4.500
66	23.8	1.375	222	268.8	4.625
72	28.1	1.500	228	283.5	4.750
78	33.2	1.625	234	298.6	4.875
84	38.5	1.750	240	314.2	5.000
90	44.2	1.875	246	330.1	5.125
96	50.3	2.000	252	346.4	5.250
102	56.8	2.125	258	363.1	5.375
108	63.6	2.250	264	380.1	5.500
114	70.9	2.375	270	397.6	5.625
120	78.5	2.500	276	415.5	5.750
126	86.6	2.625	282	433.7	5.875
132	95.0	2.750	288	452.4	6.000
138	103.9	2.875	294	471.4	6.125
144	113.1	3.000	300	490.9	6.250
150	122.7	3.125			

Table 4.11

Area and hydraulic radius for corrugated steel pipe arches flowing full

Corrugations 2 2/3 x 1/2 in.				Corrugations 3 x 1 in. and 5 x 1 in.			
Diameter in.	Pipe Arch Equivalent Size in.	Waterway Area ft^2	Hydraulic Radius $A/\pi D$ ft	Diameter in.	Pipe Arch Equivalent Size in.	Waterway Area ft^2	Hydraulic Radius $A/\pi D$ ft
15	17 x 13	1.1	0.280	54	60 x 46	15.6	1.104
18	21 x 15	1.6	0.340	60	66 x 51	19.3	1.230
21	24 x 18	2.2	0.400	66	73 x 55	23.2	1.343
24	28 x 20	2.9	0.462	72	81 x 59	27.4	1.454
30	35 x 24	4.5	0.573	78	87 x 63	32.1	1.573
36	42 x 29	6.5	0.690	84	95 x 67	37.0	1.683
42	49 x 33	8.9	0.810	90	103 x 71	42.4	1.800
48	57 x 38	11.6	0.924	96	112 x 75	48.0	1.911
54	64 x 43	14.7	1.040	102	117 x 79	54.2	2.031
60	71 x 47	18.1	1.153	108	128 x 83	60.5	2.141
66	77 x 52	21.9	1.268	114	137 x 87	67.4	2.259
72	83 x 57	26.0	1.380	120	142 x 91	74.5	2.373

Table 4.12

Area and hydraulic radius for structural plate pipe arches
(6x2 corrugation, 3N corner plates with 18 in. radius) flowing full

Dimensions, ft - in.		Waterway Area	Hydraulic Radius
Span	Rise	ft^2	ft
6-1	4-7	22	1.29
6-4	4-9	24	1.35
6-9	4-11	26	1.39
7-0	5-1	28	1.45
7-3	5-3	30	1.51
7-8	5-5	33	1.55
7-11	5-7	35	1.61
8-2	5-9	38	1.67
8-7	5-11	40	1.71
8-10	6-1	43	1.77
9-4	6-3	45	1.81
9-6	6-5	48	1.87
9-9	6-7	51	1.93
10-3	6-9	54	1.97
10-8	6-11	57	2.01
10-11	7-1	60	2.07
11-5	7-3	63	2.11
11-7	7-5	66	2.17
11-10	7-7	70	2.23
12-4	7-9	73	2.26
12-6	7-11	77	2.32
12-8	8-1	81	2.38
12-10	8-4	85	2.44
13-5	8-5	88	2.48
13-11	8-7	91	2.52
14-1	8-9	95	2.57
14-3	8-11	100	2.63
14-10	9-1	103	2.67
15-4	9-3	107	2.71
15-6	9-5	111	2.77
15-8	9-7	116	2.83
15-10	9-10	121	2.89
16-5	9-11	125	2.92
16-7	10-1	130	2.98

Table 4.13

Area and hydraulic radius for structural plate pipe arches
(6x2 corrugation, 5N corner plates with 31 in. radius) flowing full

Span ft - in.	Rise ft - in.	Area ft^2	Hydraulic Radius ft
13-3	9-4	97	2.68
13-6	9-6	102	2.74
14-0	9-8	105	2.78
14-2	9-10	109	2.83
14-5	10-0	114	2.90
14-11	10-2	118	2.94
1 5-4	10-4	123	2.98
15-7	10-6	127	3.04
15-10	10-8	132	3.10
16-3	10-10	137	3.14
16-6	1 1-0	142	3.20
1 7-0	1 1-2	146	3.24
17-2	1 1-4	151	3.30
17-5	1 1-6	157	3.36
17-11	11-8	161	3.40
18-1	11-10	167	3.45
18-7	12-0	172	3.50
18-9	12-2	177	3.56
19-3	12-4	182	3.59
19-6	12-6	188	3.65
19-8	12-8	194	3.71
19-11	12-10	200	3.77
20-5	13-0	205	3.81
20-7	13-2	211	3.87

Table 4.14

Area and hydraulic radius for structural plate arches flowing full

Dimensions[1]		Waterway Area	Wetted Perimeter	Hydraulic Radius
Span, ft	Rise, ft	ft^2	ft	ft
6.0	1-9-1/2	8	13.2	0.606
	2-3-1/2	10	14.0	0.714
	3-2	15	15.6	0.962
7.0	2-4	12	15.8	0.759
	2-10	15	16.6	0.904
	3-8	20	18.2	1.099
8.0	2-11	17	18.4	0.924
	3-4	20	19.2	1.042
	4-2	26	20.8	1.250
9.0	2-11	19	20.2	0.941
	3-10-1/2	27	21.8	1.239
	4-8-1/2	34	23.4	1.453

Table 4.14 continued

Area and hydraulic radius for structural plate arches flowing full

Dimensions[1]		Waterway Area	Wetted Perimeter	Hydraulic Radius
Span, ft	Rise, ft	ft^2	ft	ft
10.0	3-5-1/2	26	22.8	1.140
	4-5	34	24.4	1.393
	5-3	41	26.0	1.577
11.0	3-6	28	24.6	1.138
	4-5-1/2	37	26.2	1.412
	5-9	50	28.6	1.748
12.0	4-0-1/2	35	27.2	1.287
	5-0	45	28.8	1.563
	6-3	59	31.2	1.891
13.0	4-1	38	29.0	1.310
	5-1	49	30.6	1.601
	6-9	70	33.8	2.071
14.0	4-7-1/2	47	31.6	1.487
	5-7	58	33.2	1.747
	7-3	80	36.4	2.198
15.0	4-7-1/2	50	33.4	1.497
	5-8	62	35.0	1.771
	6-7	75	36.6	2.049
	7-9	92	39.0	2.359
16.0	5-2	60	36.0	1.667
	7-1	86	39.2	2.194
	8-3	1 05	41.6	2.524
17.0	5-2-1/2	63	37.8	1.667
	7-2	92	41.0	2.244
	8-10	119	44.2	2.692
18.0	5-9	74	40.4	1.832
	7-8	104	43.6	2.385
	8-11	125	46.8	2.671
19.0	6-4	87	43.0	2.023
	8-2	118	46.2	2.554
	9-5-1/2	140	49.4	2.834
20.0	6-4	91	45.6	1.996
	8-3-1/2	124	48.0	2.521
	10-0	157	51.2	3.066
21.0	6-11	104	47.4	2.194
	8-10	140	50.6	2.767
	10-6	172	53.8	3.197
22.0	7-11	128	48.7	2.628
	8-11	1 46	52.4	2.786
	11-0	190	56.4	3.369
23.0	8-0	134	52.6	2.548
	9-10	170	55.8	3.047
	11 -6	207	59.0	3.508
24.0	8-6	149	55.2	2.699
	10-4	188	58.4	3.219
	12-0	226	61.6	3.669
25.0	8-6-1/2	155	57.0	2.719
	10-10-1/2	207	61.0	3.393
	12-6	245	64.2	3.816

1 Dimensions are to inside crests and are subject to manufacturing tolerances.

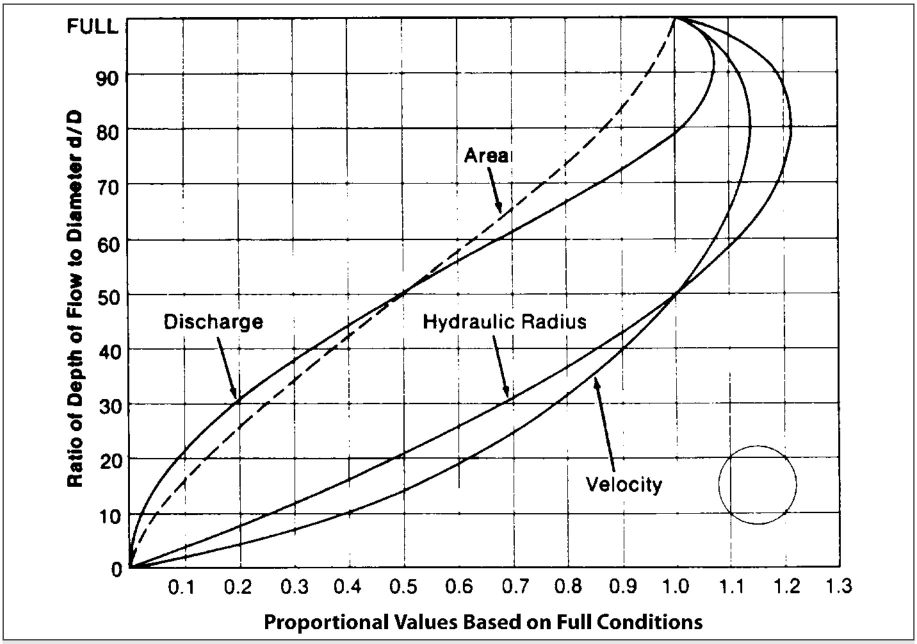


Figure 4.32 Hydraulic section parameters for circular corrugated steel and structural plate pipes.

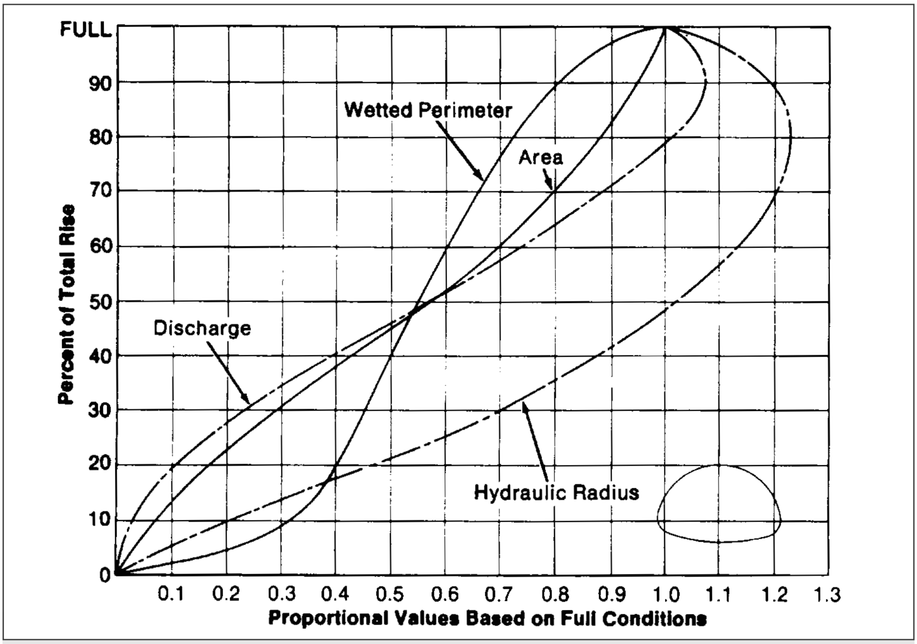


Figure 4.33 Hydraulic section parameters for corrugated steel and structural plate pipe arches.

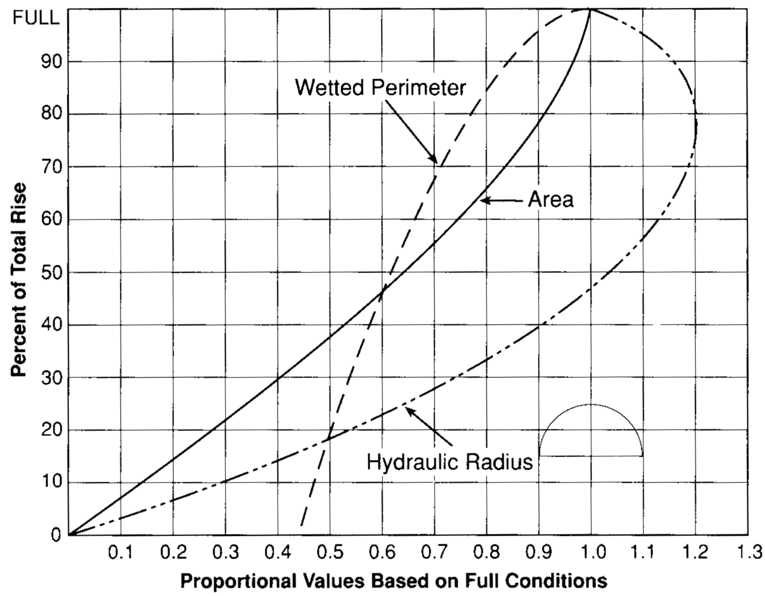


Figure 4.34 Hydraulic section parameters for structural plate arches.

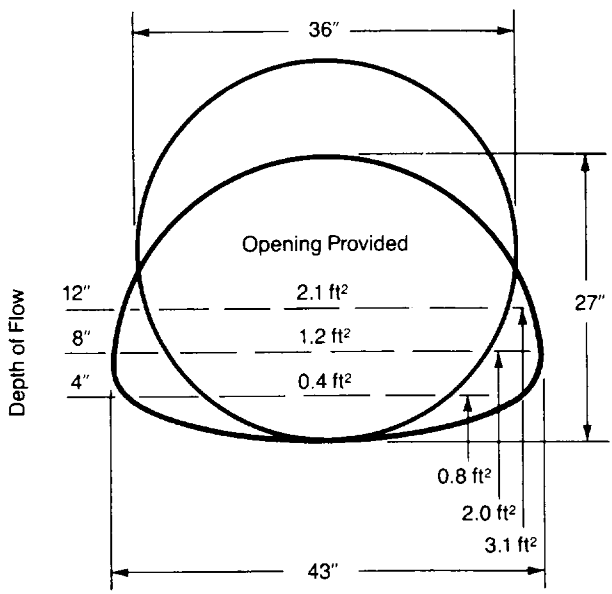


Figure 4.35 Comparison of waterway cross-sectional areas, at a constant depth of flow, in pipe and pipe arch shapes.

Hydraulic Programs

Numerous computer programs now exist to aid in the design and analysis of highway culverts. These programs possess distinct advantages over traditional hand calculation methods. The increased accuracy of programmed solutions represents a major benefit over the inaccuracies inherent in the construction and use of tables and nomographs. In addition, programmed solutions are less time consuming. This feature allows the designer to compare alternative sizes and inlet configurations very rapidly so that the final culvert selection can be based on economics. Interactive capabilities in some programs can be utilized to change certain input parameters or constraints and analyze their effects on the final design. Familiarity with culvert hydraulics and the traditional analytical methods provides a solid basis for designers to take advantage of the speed, accuracy and increased capabilities available in culvert hydraulics programs.

Most programs analyze the performance of a given culvert, although some are capable of design. Generally, the desired result of either type of program is to obtain a culvert design which satisfies hydrologic needs and site conditions by considering both inlet and outlet control. Results usually include the barrel size, inlet dimensions, headwater depth, outlet velocity and other hydraulic data. Some programs are capable of analyzing side-tapered and slope-tapered inlets. The analysis or design of the barrel size can be for one barrel only or for multiple barrels.

Some programs may contain features such as backwater calculations, performance curves, hydrologic routines and capabilities for routing based on upstream storage considerations.

HYDRAULICS OF LONG SPAN STRUCTURES

Introduction

Standard procedures are presented here to determine the headwater depth resulting from a given flow through a long span structure under both inlet and outlet control conditions. The most common long span hydraulic shapes are the horizontal ellipse, the low profile arch and the high profile arch. Useful hydraulic data pertaining to these shapes are presented in tabular and graphic form. Basic hydraulic equations, flow conditions and definitions have been given previously. However, long span hydraulics include factors which are not considered in the earlier calculations.

Design

Long span structures are often small bridges that span the flood channel. This type of structure ordinarily permits little or no ponding at the inlet. Maximum headwater is usually below the top of the structure. In other words, there is usually some freeboard between the water surface and the top of the structure. This condition is quite different from the ordinary culvert, which normally presents a small opening in an embankment crossing a larger flood channel.

The typical long span hydraulic conditions just described maintain effective approach velocity. The following long span hydraulic design procedure considers this approach velocity. The formulas and coefficients taken from the U.S. Federal Highway Administration (FHWA) methodology have been modified to include the approach velocity. In this discussion, headwater, HW, refers to the water surface and not to the energy grade line. This is different from the FHWA procedures, where HW refers to the energy grade line, which corresponds to HW + Φ in this discussion.

Design Chart

Inlet control is expected to govern in most long spans. Figure 4.36 allows the designer to conveniently calculate the headwater depth for three standard shapes having the most typical inlet condition. This figure is a plot of the two design equations below (for unsubmerged and submerged inlets), and is based on an inlet that is either a square end with a headwall or a step-beveled end with a concrete collar (Type 1 in Table 4.15). The accuracy of the curves is within the degree to which the graph can be read. Using the design discharge and the structure span and rise, the curve for the structure desired gives the ratio of the headwater depth, approach velocity head and slope correction to the structure rise. The headwater depth is determined by subtracting the velocity head and slope correction from the product of the ratio and the structure rise. Figure 4.36 also includes a table of velocity heads for a variety of approach velocities.

Table 4.15

Entrance loss coefficients for long spans

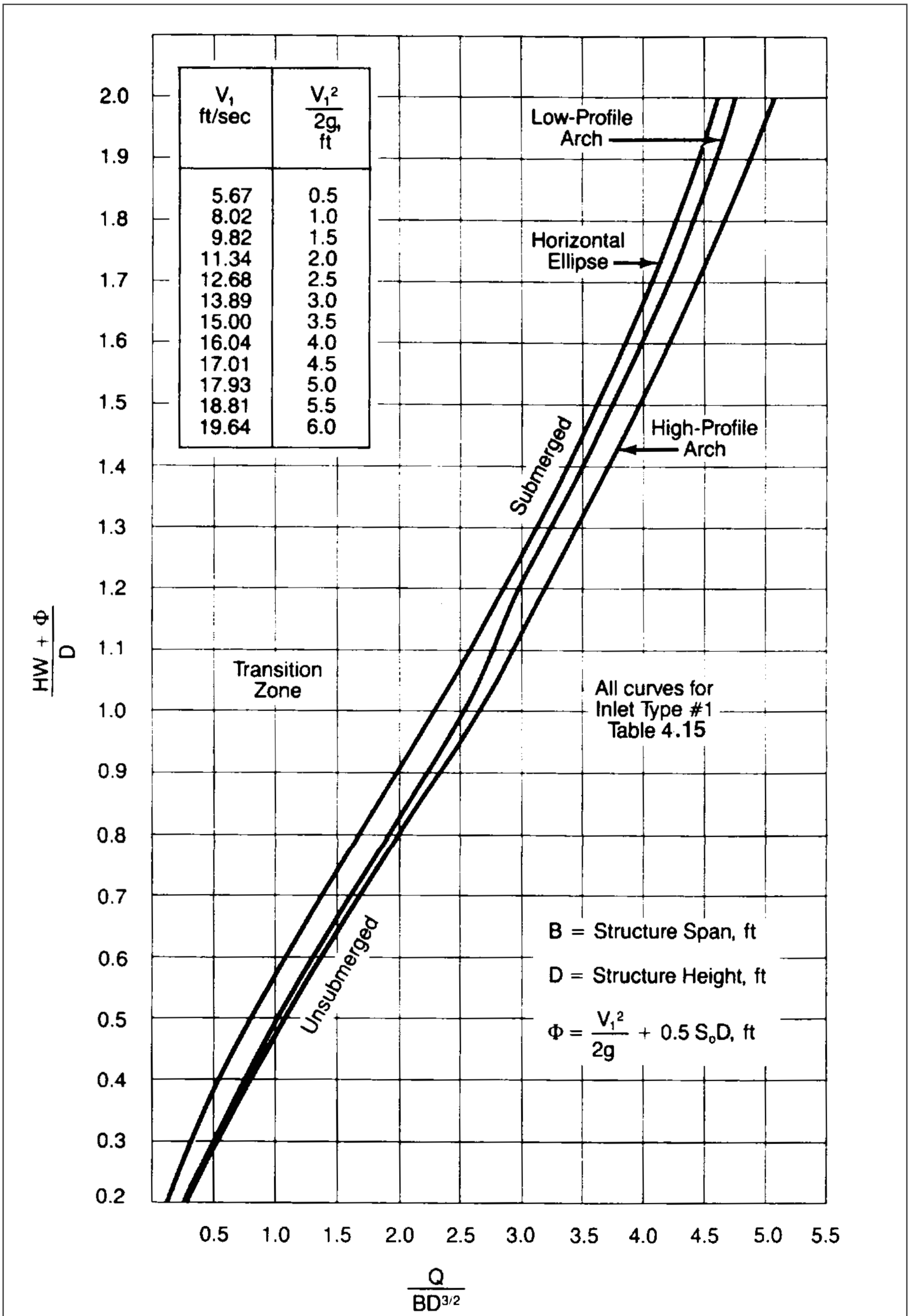
Type Inlet	Entrance Coefficients					$\frac{Q}{AD^{1/2}}$	
	k_d	k_p	k	j	k_e	Unsubmerged Maximum	Submerged Mnimum
1	0.0379	0.69	0.0083	2.0	0.5	3.3	3.8
2	0.0300	0.74	0.0018	2.5	0.2	3.3	4.2

1) Type 1 inlet is square end with headwall or step-beveled end with concrete collar.

2) Type 2 inlet is square or step-beveled end with mitered edge on headwall. Step-beveled inlets were not included in FHWA criteria.

3) Special improved inlet configurations can reduce headwater depths.

4) Coefficient k and k_d are not dimensionless.



■ **Figure 4.36** Headwater depth for long span corrugated steel structures under inlet control.

Design Calculations

Inlet Control

The equations for calculating headwater depth for long span structures under inlet control are as follows:

For unsubmerged inlets:

$$HW = H_c + H_e - 0.5\ S_o\ D - \frac{V_1^{\ 2}}{2g}$$

For submerged inlets:

$$HW = k_d\ D \left\{\frac{Q}{AD^{1/2}}\right\}^2 + k_p\ D - 0.5\ S_o\ D - \frac{V_1^{\ 2}}{2g}$$

where: HW = headwater depth from the invert to the water surface, ft
H_c = critical head, ft
H_e = increment of head above the critical head, ft
S_o = slope of the structure, ft/ft
D = rise of the structure, ft
V_1 = approach velocity, ft/s
g = gravitational constant, 32.2 ft/s^2
k_d, k_p = coefficients based on inlet type (Table 4.15)
Q = design discharge, ft^3/s
A = full cross-sectional end area of the structure, ft^2

To determine if the flow condition is submerged or unsubmerged, the value of $\left\{\frac{Q}{AD^{1/2}}\right\}^2$ is calculated and reference is made to Table 4.15. If the flow is in the transition zone between unsubmerged and submerged, a reasonable approximation can be made by using both equations and interpolating based on where the value occurs relative to the limits in the table. When a performance curve is plotted, such as in Figure 4.36, the transition zone is filled in manually.

The critical head is equal to the critical depth in the structure at design flow plus the velocity head at that flow:

$$H_c = d_c + \frac{V_c^{\ 2}}{2g}$$

where: d_c = critical depth, ft
V_c = critical velocity, ft/s

The critical depth can be interpolated from Tables 4.16 through 4.18. Using the design discharge, the critical depth (as a decimal fraction of the structure rise) is estimated by interpolating between known discharges for a number of set critical depth decimal fractions.

Table 4.16

Hydraulic section parameters for long span structural plate horizontal ellipses

Span x Rise (B x D) ft - in.	Full Flow Data				Discharge - (Q), ft^2/sec					
					Critical Depth Factor*					
	Area ft^2	WP ft	R ft	$AR^{2/3}$	0.40	0.50	0.60	0.70	0.80	0.90
19-4 x 12-9	191	50.7	3.77	462.7	769	1204	1714	2316	3083	4183
20-1 x 13-0	202	52.3	3.86	497.1	823	1282	1832	2478	3298	4502
20-2 x11-11	183	50.7	3.61	430.6	708	1110	1584	2153	2871	3935
20-10 x 12-2	194	52.3	3.71	464.9	756	1154	1694	2298	3088	4194
21-0 x 15-2	248	57.1	4.35	660.9	1073	1684	2390	3225	4336	5901
21-11 x 13-1	221	55.5	3.98	555.0	897	1403	2005	2725	3640	4966
22-6 x 15-8	274	60.3	4.55	752.4	1228	1921	2732	3687	4886	5979
23-0 x 14-1	249	58.7	4.25	653.3	1051	1645	2347	3185	4256	5801
23-3 x 15-11	288	61.9	4.65	802.3	1298	2033	2889	3903	5179	7046
24-4 x 16-11	320	65.1	4.92	925.7	1486	2327	3307	4464	5914	8055
24-6 x 14-8	274	61.9	4.43	739.1	1177	1843	2634	3577	4785	6518
25-2 x 14-11	287	63.5	4.53	785.7	1242	1947	2782	3780	5060	6881
25-5 x 16-9	330	66.7	4.95	958.5	1523	2383	3391	4588	6101	8295
26-1 x 18-2	369	69.9	5.28	1118.9	1775	2778	3949	5331	7046	9611
26-3 x 15-10	320	66.9	4.80	910.6	1430	2240	3196	4340	5804	7902
27-0 x 16-2	334	68.3	4.89	962.2	1503	2356	3366	4572	6113	8303
27-2 x 19-1	405	73.1	5.54	1268.0	1999	3131	4448	6004	7953	10817
27-11 x 19.5	421	74.7	5.64	1334.0	2095	3278	4660	6290	8329	11325
28-1 x 17-1	369	71.5	5.16	1101.9	1714	2683	3830	5192	6943	9438
28-10 x 17-5	384	73.1	5.26	1161.4	1795	2812	4016	5452	7288	9919
29-5 x 19-11	455	77.9	5.84	1475.5	2289	3587	5098	6887	9143	12434
30-1 x 20-2	472	79.5	5.94	1548.1	2391	3744	5326	7198	9563	13008
30.3 x 17-11	415	76.3	5.44	1283.6	1968	3084	4406	5985	8014	10900
31-2 x 21-2	513	82.7	6.20	1731.3	2659	4166	5925	8003	10622	14429
31-4 x 18-11	454	79.5	5.71	1450.4	2212	3467	4950	6720	8983	12205
32-1 x 19-2	471	81.1	5.81	1522.2	2309	3617	5173	7020	9389	12774
32-3 x 22-2	555	85.9	6.46	1925.1	2947	4615	6561	8870	11752	15964
33-0 x 22-5	574	87.5	6.56	2011.5	3064	4798	6825	9220	12236	16639
33-2 x 20-1	512	84.3	6.08	1705.6	2577	4038	5762	7819	10451	14210
34-1 x 23-4	619	90.7	6.82	2226.1	3376	5286	7495	10150	13464	18280
34-7 x 20-8	548	87.5	6.26	1861.4	2792	4372	6245	8481	11320	15424
34-11 x 21-4	574	89.1	6.44	1986.9	2975	4661	6652	9070	12053	16397
35-1 x 24-4	665	93.9	7.08	2452.0	3705	5801	8246	11131	14754	20048
36-0 x 22-4	619	92.3	6.71	2202.1	3286	5146	7341	9950	13283	18054
37-2 x 22-2	631	93.9	6.72	2247.0	3328	5215	7450	10100	13537	18381

* Multiply factor by structure rise.

Table 4.17

Hydraulic section parameters for long span structural plate low profile arches

Span x Rise (B x D) ft - in.	Full Flow Data				Discharge - (Q), ft^3/sec					
					Critical Depth Factor*					
	Area ft^2	WP ft	R ft	$AR^{2/3}$	0.4D	0.5D	0.6D	0.7D	0.8D	0.9D
20-1 x 7-6	120	47.9	2.51	223	579	819	1119	1443	1839	2448
19-5 x 6-9	105	45.6	2.30	183	480	681	933	1206	1532	2049
21-6 x 7-9	133	51.0	2.62	253	656	929	1266	1632	2081	2723
22-3 x 7-11	140	52.5	2.67	269	698	986	1344	1720	2207	2926
23-0 x 8-0	147	54.1	2.72	286	738	1042	1420	1829	2324	3083
23-9 x 8-2	154	55.6	2.77	304	784	1112	1508	1944	2466	3267
24-6 x 8-3	161	57.2	2.82	321	830	1180	1593	2054	2612	3422
25-2 x 8-5	168	58.7	2.86	339	876	1249	1682	2166	2756	3640
25-11 x 8-7	176	60.2	2.91	359	923	1320	1774	2290	2901	3848
27-3 x 10-0	217	64.8	3.34	485	1228	1733	2340	3017	3836	5046
28-1 x 9-6	212	65.6	3.23	463	1179	1666	2252	2901	3685	4858
28-9 x 10-3	234	67.9	3.44	533	1343	1896	2558	3297	4193	5538
28-10 x 9-8	220	67.1	3.28	486	1236	1747	2361	3040	3863	5105
30-3 x 9-11	237	70.2	3.38	534	1353	1914	2586	3326	4220	5543
30-11 x 10-8	261	72.5	3.59	612	1536	2168	2922	3761	4760	6292
31-7 x 12-1	309	76.1	4.06	786	1901	2684	3607	4676	5961	7836
31-0 x 10-1	246	71.7	3.43	560	1416	2004	2702	3476	4411	5806
32-4 x 12-3	319	77.6	4.11	819	1979	2795	3760	4869	6201	8142
31-9 x 10-2	255	73.3	3.47	585	1480	2099	2824	3631	4611	6086
33-1 x 12-5	330	79.1	4.17	855	2085	2907	3916	5065	6443	8500
33-2 x 11-1	289	77.1	3.74	696	1741	2459	3313	4264	5413	7099
34-5 x 13-3	367	83.0	4.42	988	2372	3346	4467	5815	7400	9730
34-7 x 11-4	308	80.2	3.84	755	1886	2674	3589	4614	5853	7722
37-11 x 15-7	477	92.9	5.13	1419	3356	4726	6290	8158	10390	13646
35-4 x 11-5	318	81.7	787	1962	2784	3730	4782	6089	8015	
38-8 x 15-9	490	94.4	1469	3466	4881	6496	8426	10727	14122	

* Multiply factor by structure rise.

Housing Development in Thornton, Colorado. Super Cor Box Culvert 35'-9" span x 7'-9" rise.

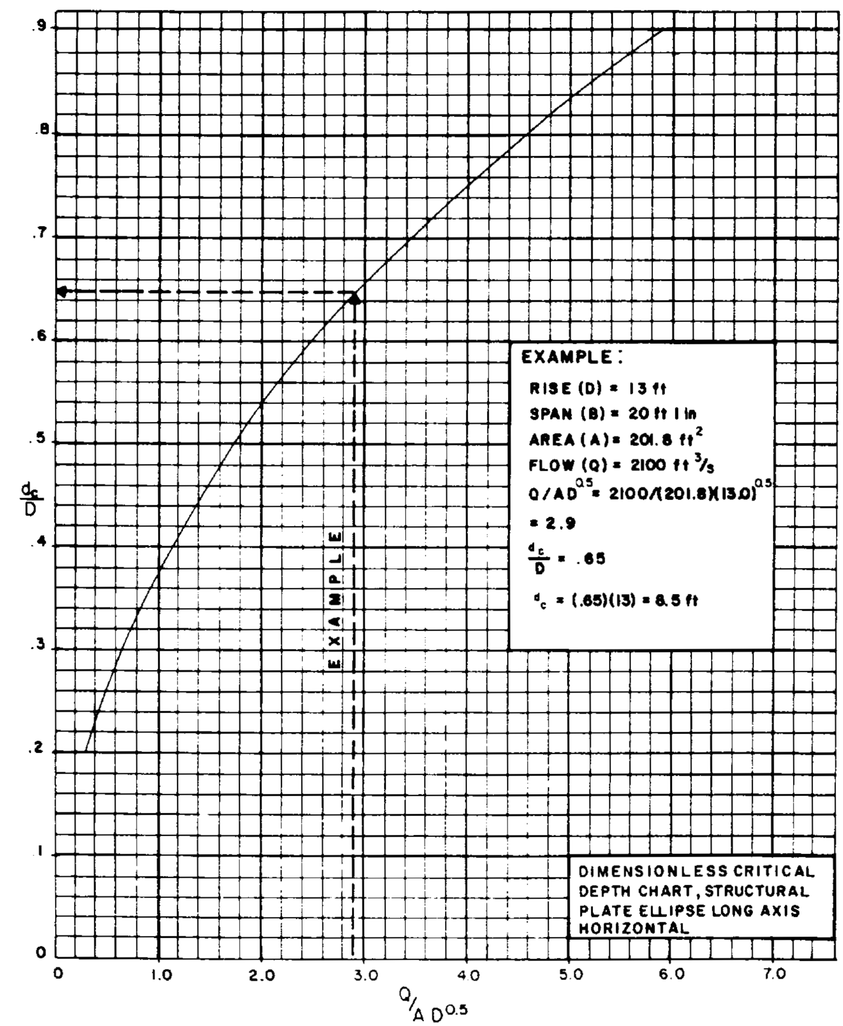
Table 4.18

Hydraulic section parameters for long span structural plate high profile arches

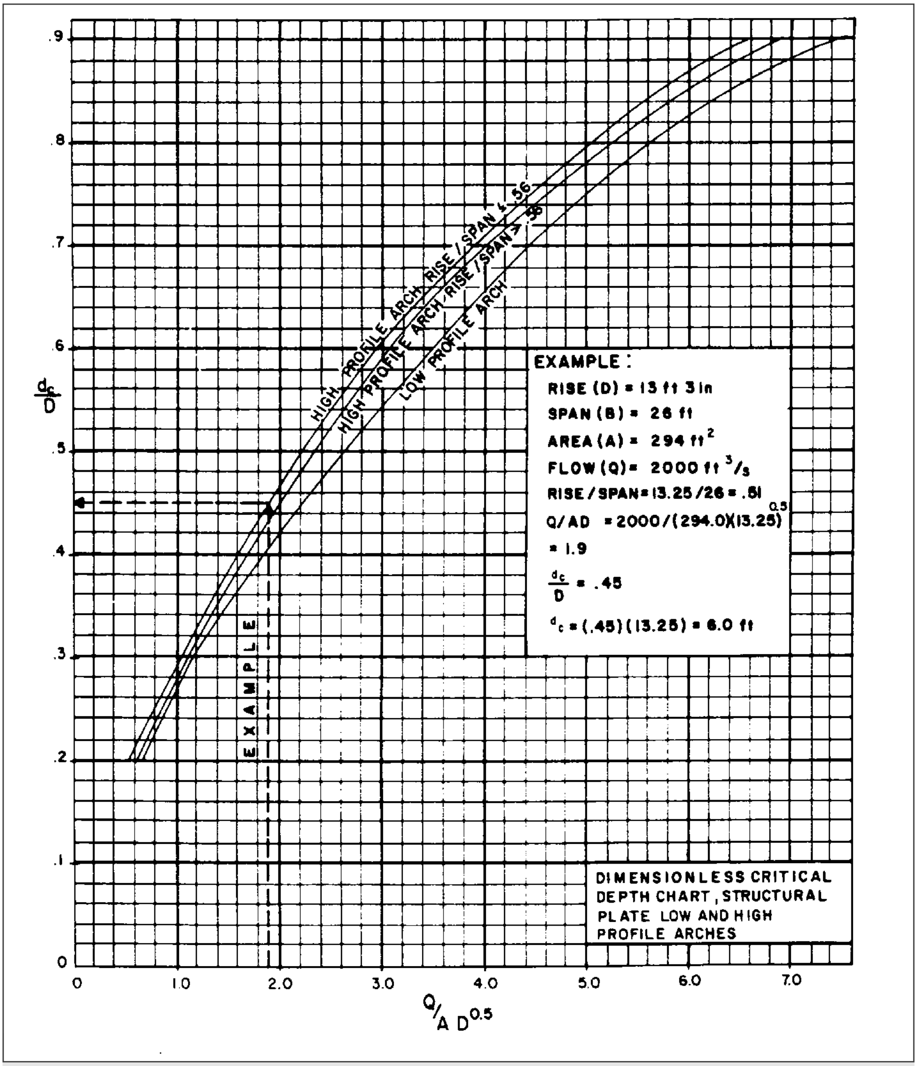
Span x Rise (B x D) ft - in.	Full Flow Data				Discharge - (Q), ft^3/sec					
					Critical Depth Factor*					
	Area ft^2	WP ft	R ft	AR $^{2/3}$	0.4D	0.5D	0.6D	0.7D	0.8D	0.9D
20-1 x 9-1	152	50.8	2.99	315.5	785	1107	1480	1923	2466	3282
20-8 x 12-1	214	56.5	3.78	518.6	1191	1687	2264	2936	3790	5044
21-6 x 11-8	215	57.5	3.73	516.1	1179	1669	2234	2911	3765	4989
22-10 x 14-6	284	63.9	4.45	768.9	1690	2402	3227	4193	5412	7209
22-3 x 11-10	224	59.1	3.80	546.5	1246	1762	2361	3077	3974	5297
22-11 x 14-0	275	63.3	4.34	731.3	1601	2279	3050	3969	5140	6853
23-0 x 11-11	234	60.7	3.86	576.7	1315	1858	2491	3246	4191	5589
24-4 x 14-10	309	67.2	4.60	854.4	1874	2658	3569	4636	5989	7980
23-9 x 12-1	244	62.3	3.93	608.8	1385	1956	2623	3418	4417	5869
24.6 x 13-9	288	66.0	4.37	770.5	1680	2376	3187	4154	5391	7200
25-9 x 15-1	334	70.5	4.74	942.5	2063	2924	3923	5096	6586	8769
25-2 x 13-1	283	66.6	4.25	742.0	1650	2331	3125	4079	5280	7030
26-6 x 15-3	347	72.1	4.81	988.1	2161	3062	4106	5312	6896	9184
25-11 x 13-3	294	68.2	4.31	778.4	1730	2445	3276	4280	5534	7348
27-3 x 15-5	360	73.7	4.88	1034.6	2260	3201	4292	5577	7803	9584
27-5 x 13-6	317	71.3	4.44	855.0	1896	2679	3591	4692	6064	8068
29-5 x 16-5	412	79.2	5.20	1235.4	2697	3820	5118	6639	8570	11390
28-2 x 14-5	348	74.0	4.70	976.0	2123	2998	4019	5255	6802	9050
30-1 x 18-0	466	82.8	5.63	1474.0	3111	4402	5920	7694	9952	13266
30-3 x 15-5	399	79.5	5.02	1169.0	2539	3589	4811	6278	8114	10775
31-7 x 18-4	496	86.1	5.77	1596.6	3366	4768	6405	8315	10760	14291
31-0 x 15-7	412	81.1	5.08	1216.8	2642	3734	5004	6534	8437	11173
31-8 x 17-9	483	85.4	5.65	1531.5	3222	4556	6114	7960	10323	13760
32-4 x 19-11	553	90.0	6.18	1863.2	3808	5404	7259	9450	12256	16350
31-9 x 17-2	469	84.8	5.53	1466.4	3080	4353	5836	7615	9890	13190
33-1 x 20-1	570	91.2	6.25	1934.9	3940	5610	7534	9807	12721	16963
32-6 x 17-4	484	86.4	5.60	1524.8	3200	4522	6061	7917	10270	13675
33-10 x 20-3	587	92.9	6.33	2009.9	4106	5820	7814	10172	13197	17607
34-0 x 17-8	513	89.6	5.73	1643.2	3445	4867	6524	8532	11054	14703
34-7 x 19-10	590	93.9	6.28	2007.6	4095	5797	7775	10136	13176	17575
34-8 x 17-9	528	91.2	5.79	1703.0	3572	5043	6762	8844	11458	15210
35-4 x 20-0	607	95.5	6.35	2080.4	4255	6022	8076	10534	13697	18270

* Multiply factor by structure rise.

Some of the long span culverts and special culvert shapes had no critical depth charts. These special shapes are available in numerous sizes, making it impractical to produce individual critical depth curves for each culvert size and shape. Hence, dimensionless critical depth curves, Figures 4.37 and 4.38, have been developed for the shapes which have adequate geometric relationships. It should be noted that these special shapes are not truly geometrically similar, and any generalized set of geometric relationships will involve some degree of error. The amount of error is unknown.



■ **Figure 4.37** Critical depth for horizontal ellipse long span structural plate corrugated steel bridges.

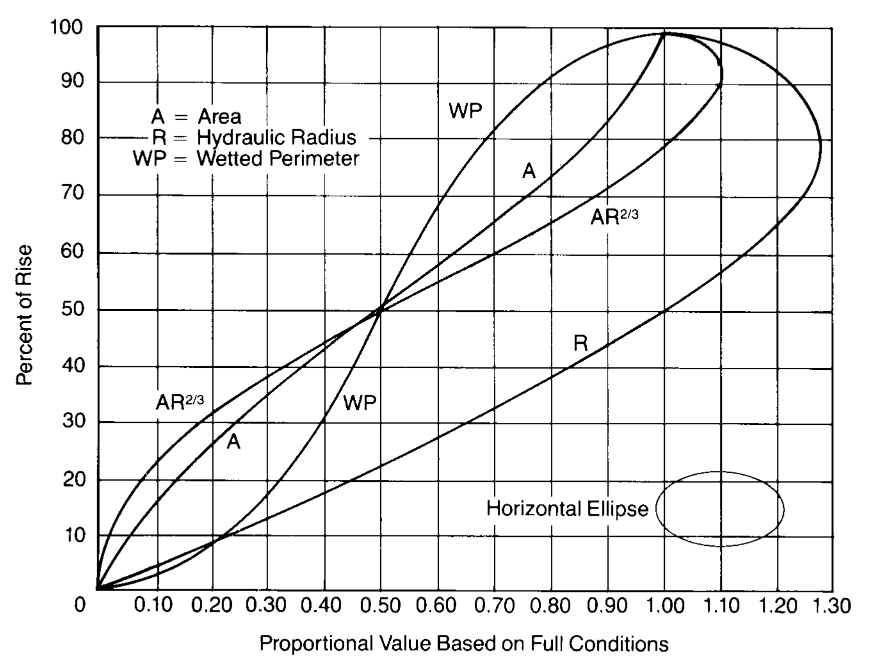


■ **Figure 4.38** Critical depth for low and high profile arch long span structural plate corrugated steel bridges.

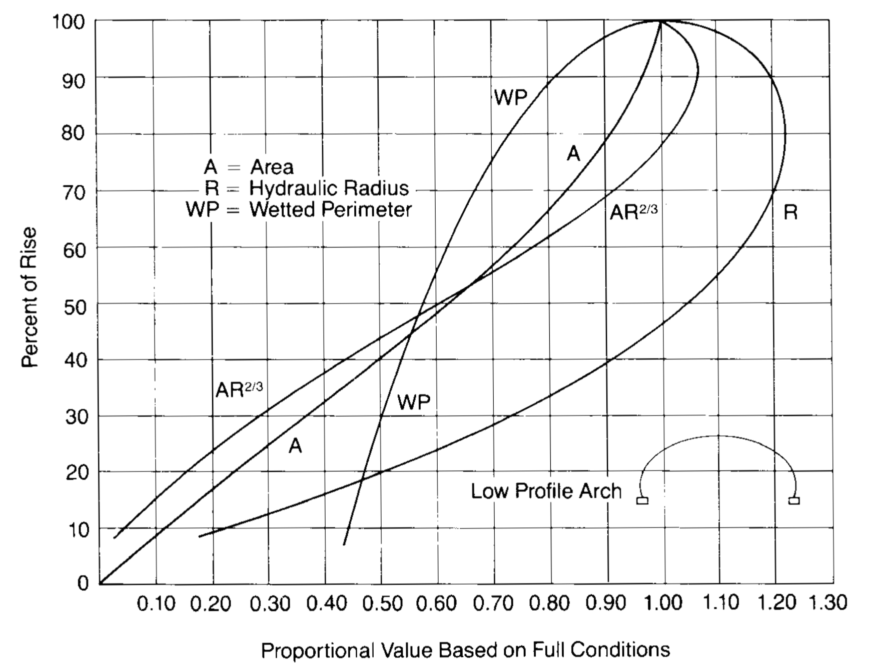
The critical velocity is calculated by dividing the design discharge by the partial flow area corresponding to the critical depth. The partial flow area can be determined from Figures 4.39 through 4.41 using the critical depth as a percentage of the structure rise. The partial flow area is the product of the proportional value from the figure and the full cross sectional area of the structure. The critical velocity is then:

$$V_c = \frac{Q}{A_c}$$

where: A_c = partial flow area based on the critical depth, ft^2



■ **Figure 4.39** Hydraulic section parameters for long span horizontal ellipses.



■ **Figure 4.40** Hydraulic section parameters for long span low profile arches.

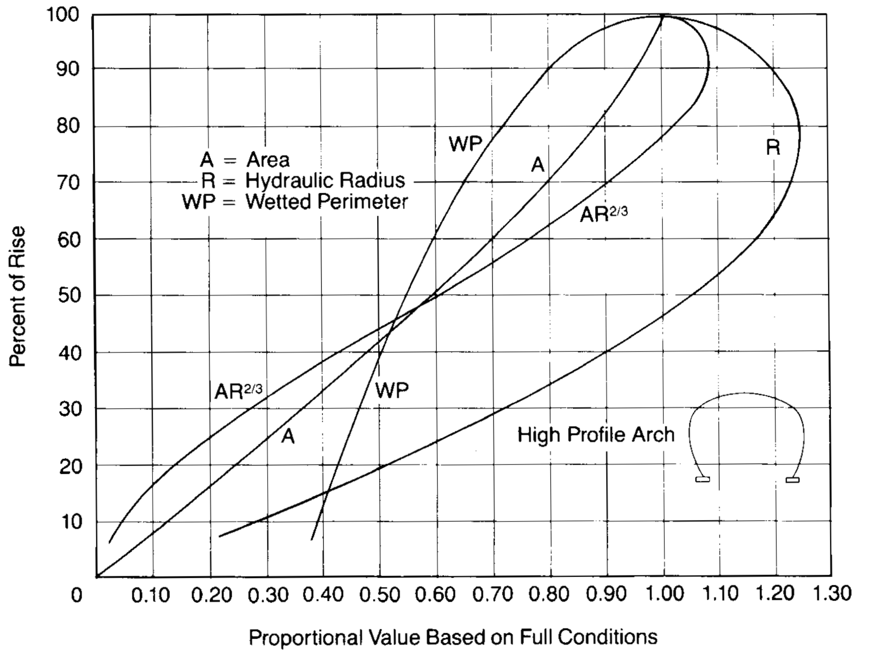


Figure 4.41 Hydraulic section parameters for long span high profile arches.

The accuracy of the critical depth may be checked using the basic equation for critical flow:

$$Q_c = \sqrt{\frac{gA_c^3}{T_c}}$$

where: T_c= width of the water surface for the critical depth case, ft

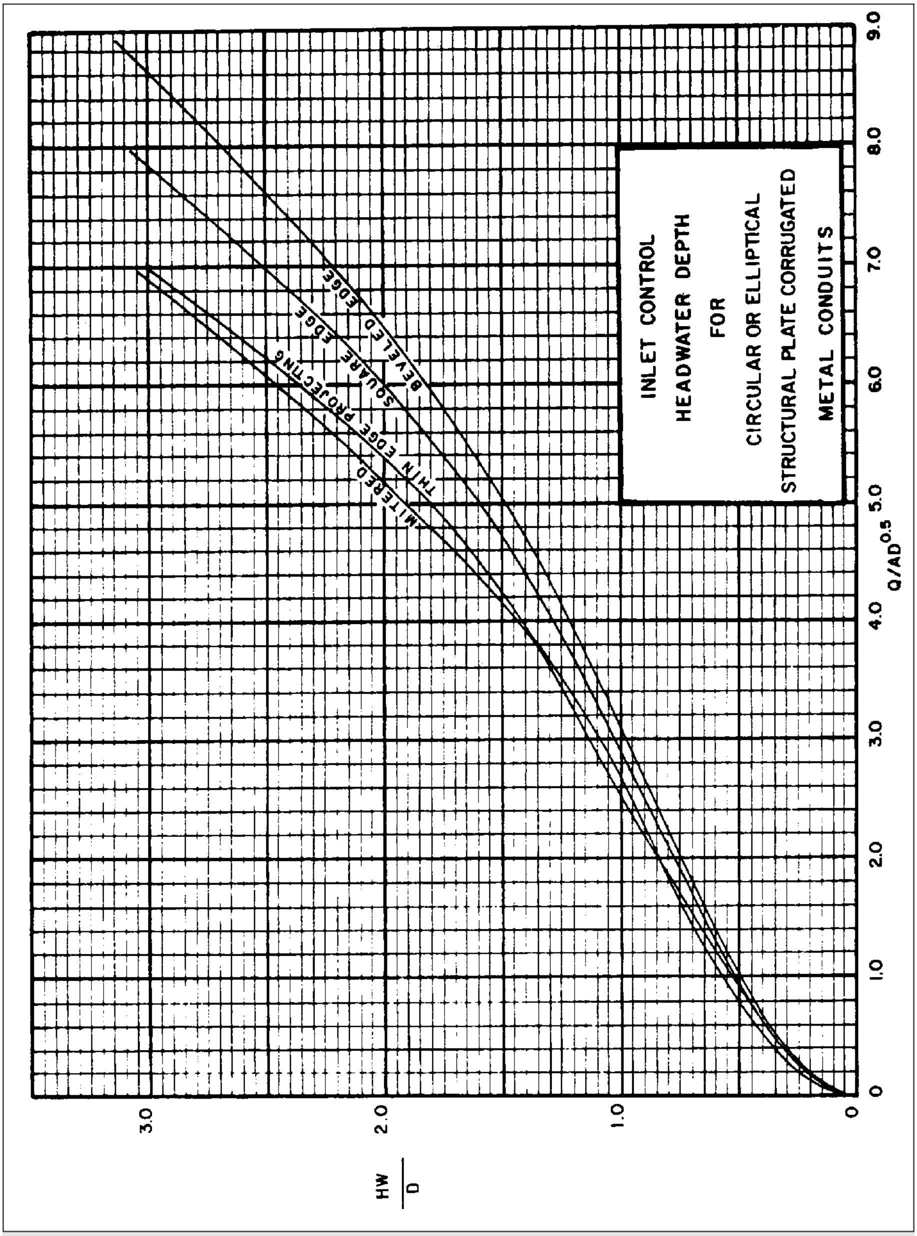
For this calculation, detailed structure cross section geometry is required in order to calculate the water surface width when the water depth is the critical depth.

The increment of head above the critical head is:

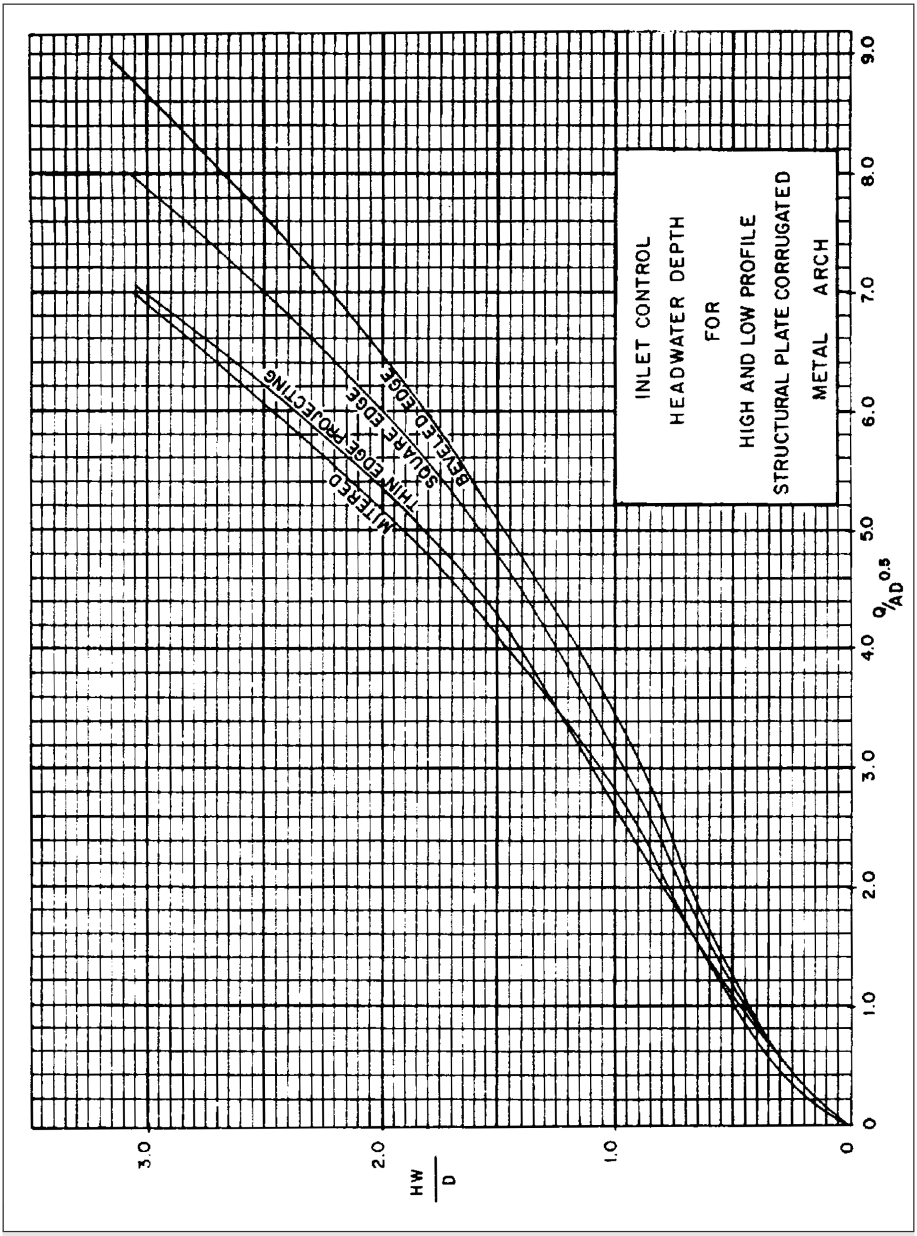
$$H_e = k\,D\left\{\frac{Q}{AD^{1/2}}\right\}^j$$

where: k, j = coefficients based on inlet type (Table 4.15)

By plotting the results of the unsubmerged and submerged calculations and connecting the resultant curves with transition lines, the dimensionless design curves shown in Figures 4.42 and 4.43 were developed. All circular and elliptical shapes can be represented by a single curve for each inlet edge configuration. A similar set of curves was developed for high and low profile arches. It is recommended that the curves shown in Figure 4.42 be used for curved shapes including circles, ellipses and pear shapes, and that the high and low profile arch curves in Figure 4.43 be used for all true arch shapes (those with a flat bottom).



■ **Figure 4.42** Headwater depth for circular and elliptical long span structural plate corrugated steel bridges.



■ **Figure 4.43** Headwater depth for high and low profile arch long span structural plate corrugated steel bridges.

Outlet Control

Free Water Surface

The situation where a long span has a free water surface extending through its full or nearly full length, as shown in Figure 4.5 D (possibly the most common flow condition), exists when the headwater depth is less than:

$$D + (1 + k_e)\frac{V_c^2}{2g}$$

where: k_e = entrance loss coefficient based on inlet type (Table 4.15)

Under this condition, the headwater depth must be determined by a backwater analysis if accurate results are required. Datum points d1 and d2 are established upstream and downstream from the structure, beyond the influence of the entrance and outlet. The backwater analysis determines the water surface profile by starting at the downstream point and moving to the upstream point. The backwater analysis must consider channel geometry between the downstream point and the outlet end of the structure, outlet loss, changing geometry of flow within the structure, inlet loss, and conditions between the inlet end of the structure and the upstream point.

As discussed previously, long span hydraulic properties are provided in Tables 4.16 through 4.18 and Figures 4.39 through 4.41, and entrance loss coefficients are in Table 4.15. The exit loss for these types of structures is typically very small and is often assumed to be zero.

Backwater analyses are considered outside the scope of this design manual. There are references that provide guidance for this procedure. In particular, the FHWA's "Hydraulic Design of Highway Culverts" CDROM contains a discussion and example of the backwater analysis procedure.

Full Flow

When full flow or nearly full flow exists, the headwater depth is determined by the following:

$$HW = \left(k_e + \frac{2gn^2 L}{R^{4/3}} + 1\right)\frac{V^2}{2g} + h_o - L\,S_o - \frac{V_1^2}{2g}$$

where:
- HW = headwater depth, ft
- k_e = entrance loss coefficient (Table 4.15)
- g = gravitational constant = 32.2 ft/s^2
- n = Manning's friction factor (Table 4.7)
- L = length of long span, ft
- R = hydraulic radius, ft = A / WP

A = full cross sectional area of the long span, ft^2
WP = perimeter of the long span, ft
V = velocity, ft/s
h_o = outlet datum, ft
S_o = slope of structure, ft/ft
V_1 = approach velocity, ft/s

These conditions are as shown in Figure 4.5 A through C. They occur when the head-water depth is greater than:

$$D + (1 + k_e)\frac{V_c^2}{2g}$$

For arches or lined structures, a composite Manning's n value must be developed. A method described in an FHWA document is based on the assumption that the conveyance section can be broken down into a number of parts with associated wetted perimeters and Manning's n values. Each part of the conveyance section is then assumed to have a mean velocity equal to the mean velocity of the entire flow section. These assumptions lead to:

$$n = \left[\frac{\sum_{i=1}^{G}(p_i n_i^{1.5})}{p}\right]^{0.67}$$

where: n = weighted Manning's n value
G = number of different roughnesses in the perimeter
p_i = wetted perimeter influenced by material i, ft
n_i = Manning's n value for material i
p = total wetted perimeter, ft

In the case of arches, the wetted perimeter used in hydraulic radius calculations includes that portion of the structure above the natural channel and the natural channel itself.

For flow conditions as shown in Figure 4.5 A and B, when the tailwater depth is equal to or greater than the structure rise:

$$h_o = TW$$

For flow conditions as shown in Figure 4.5 C, when the tailwater depth is less than the structure rise:

$$h_o = \frac{d_c + D}{2} \quad \text{or TW (whichever is greater)}$$

The velocity, V, is determined by dividing the design discharge by the area, where the area is the full cross sectional area of the long span structure.

The remaining terms in the equation can be determined as previously discussed.

Summary of Procedure

Step 1. Collect all available information for the design. This includes the required design discharge, the structure length and slope, an allowable headwater elevation or depth, the average and maximum flood velocities in the channel, the proposed entrance type and a desired structure shape.

Step 2. Select an initial structure size. This may be an arbitrary choice or estimated using a maximum allowable velocity. To estimate a structure size, the minimum structure end area is determined by dividing the design discharge by the maximum allowable velocity. Geometric constraints may also influence the choice of an initial structure size. An example of this is where a minimum structure span is required to bridge a channel.

Step 3. Use Figure 4.36 and the design parameters to obtain a value for HW + Φ and then the headwater depth, HW. When required, more accurate results can be achieved by using the inlet control formulas to calculate the headwater depth.

Step 4. Check the calculated headwater depth against the allowable headwater depth. If the calculated headwater depth is greater than the allowable, select a larger structure and repeat Step 3. If the calculated headwater depth is less than the allowable, this is the resulting headwater depth for the structure selected under inlet control.

Step 5. Calculate $D + (1 + k_e) \dfrac{V_c^2}{2g}$

If this value is greater than the allowable headwater depth, use the backwater curve method to determine the water surface profile through the structure and the headwater depth. If this value is equal to or less than the allowable headwater depth, the full flow formula should be used to determine the headwater depth. The resulting headwater depth is for the structure selected under outlet control.

Step 6. Compare the inlet and outlet control headwater depths and use the larger. If the resulting headwater depth is greater than the allowable, a larger size or different shape structure should be chosen and the procedure repeated. If the headwater depth is significantly less than the allowable, a smaller size can be chosen and the procedure repeated in order to economize on the structure size.

HYDRAULICS OF STEEL BOX CULVERTS

Where large waterway openings are required with no or minimal ponding, a box culvert is often used. With a HW ratio of less than one (1.0), the steel structural plate box culvert may be designed as an open channel. This is the most efficient hydraulic design for this condition.

By examining the geometry, it can be seen that the nearly vertical legs and flat bottom will provide a linear relationship with lower depths of flow (to 0.6D, where D is the box culvert rise). As the water surface elevation increases and begins to contact the corner or haunch sections, the wetted perimeter increases at a rate faster than the rate of increase in the waterway area. At water depths of 0.8D to 1.0D, there is a rapid increase in wetted perimeter and very little increase in area. Therefore, it can be seen that maximum flow will occur at a point somewhat less than full (0.8 to 0.9D).

Manning's equation is the accepted design method for open channel flow. Table 4.19 and Figures 4.44 through 4.57 provide hydraulic design information for steel box culverts with a 6 x 2 inch corrugation. The procedure is similar to that summarized previously.

Table 4.19

Hydraulic section parameters for structural plate box culverts

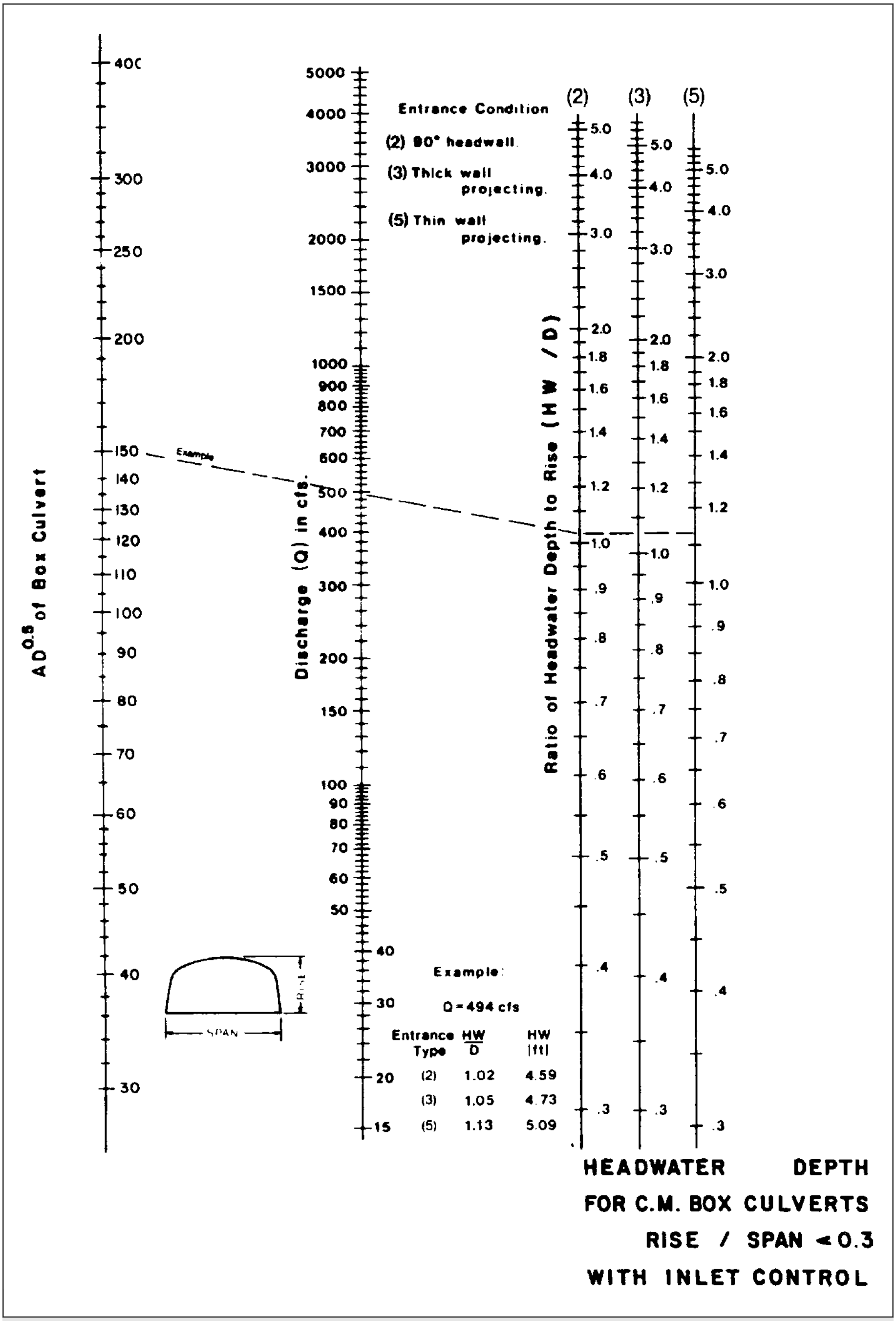
No.	Span ft-in.	Rise ft-in.	Area (sq ft)	WP ft.	$AR^{2/3}$	$AD^{1/2}$	No.	Span ft-in.	Rise ft-in.	Area (sq ft)	WP ft.	$AR^{2/3}$	$AD^{1/2}$
1	9-2	2-6	18.4	20.0	17.4	29.1	28	11-2	4-3	39.4	27.7	49.8	81.2
2	9-8	2-7	20.2	22.2	19.0	32.5	29	19-5	4-3	66.0	42.6	88.4	136.1
3	10-6	2-8	22.6	23.8	21.8	36.9	30	11-9	4-4	42.4	28.3	55.6	88.3
4	11-1	2-9	24.8	24.3	25.1	41.1	31	16-3	4-4	59.5	37.0	81.7	123.9
5	11-10	2-10	27.8	26.7	28.5	46.8	32	12-6	4-5	46.9	30.7	62.3	98.6
6	12-9	2-11	30.6	28.3	32.2	52.3	33	13-3	4-6	49.4	31.5	66.7	104.8
7	13-2	3-1	33.5	29.6	36.3	58.8	34	16-10	4-6	64.1	38.8	89.5	136.0
8	14-1	3-2	36.6	31.3	40.6	65.1	35	20-0	4-6	70.8	43.2	98.5	150.2
9	14-6	3-3	39.0	31.7	44.8	70.3	36	17-9	4-7	67.2	40.0	95.0	143.9
10	9-0	3-4	24.2	21.4	26.3	44.2	37	20-8	4-7	74.7	45.0	104.7	159.9
11	10-1	3-4	27.7	23.5	30.9	50.6	38	13-9	4-8	54.8	33.0	76.8	118.4
12	10-10	3-5	30.8	25.7	34.8	56.9	39	14-7	4-9	59.1	35.0	83.9	128.8
13	15-4	3-5	43.3	34.1	50.8	80.0	40	18-4	4-9	73.1	42.0	105.8	159.3
14	11-6	3-6	33.2	26.6	38.5	62.1	41	10-0	4-11	39.1	25.7	51.7	86.7
15	16-0	3-6	46.2	35.5	55.0	86.4	42	11-0	4-11	44.2	28.2	59.7	98.0
16	12-2	3-8	37.2	28.6	44.3	71.2	43	15-0	4-11	63.2	35.7	92.5	140.1
17	16-8	3-8	50.7	37.3	62.2	97.1	44	19-2	4-11	78.2	43.2	116.1	173.4
18	12-10	3-9	39.7	29.4	48.5	76.9	45	21-6	4-11	83.8	47.3	122.6	185.8
19	13-6	3-10	44.0	31.0	55.6	86.1	46	11-8	5-0	48.2	29.0	67.6	107.8
20	17-6	3-10	54.0	38.0	68.2	105.7	47	15-10	5-0	68.1	37.8	100.8	152.3
21	14-4	4-0	47.8	33.2	61.0	95.6	48	12-5	5-1	52.5	31.3	74.2	118.4
22	18-2	4-0	58.8	40.0	76.0	117.6	49	19-8	5-1	82.3	44.5	124.0	185.6
23	9-6	4-1	31.1	23.6	37.4	62.8	50	12-10	5-2	56.6	32.0	82.8	128.7
24	14-10	4-1	51.3	34.2	67.2	103.7	51	16-4	5-2	72.2	38.8	109.2	164.1
25	10-7	4-2	35.9	26.2	44.3	73.3	52	17-2	5-3	77.6	40.6	119.5	177.8
26	15-7	4-2	55.6	36.1	74.1	113.5	53	20-8	5-3	88.4	46.5	135.6	202.5
27	18-9	4-2	62.2	40.7	82.6	127.0	54	13-8	5-4	60.8	34.0	89.6	140.4

(continued)

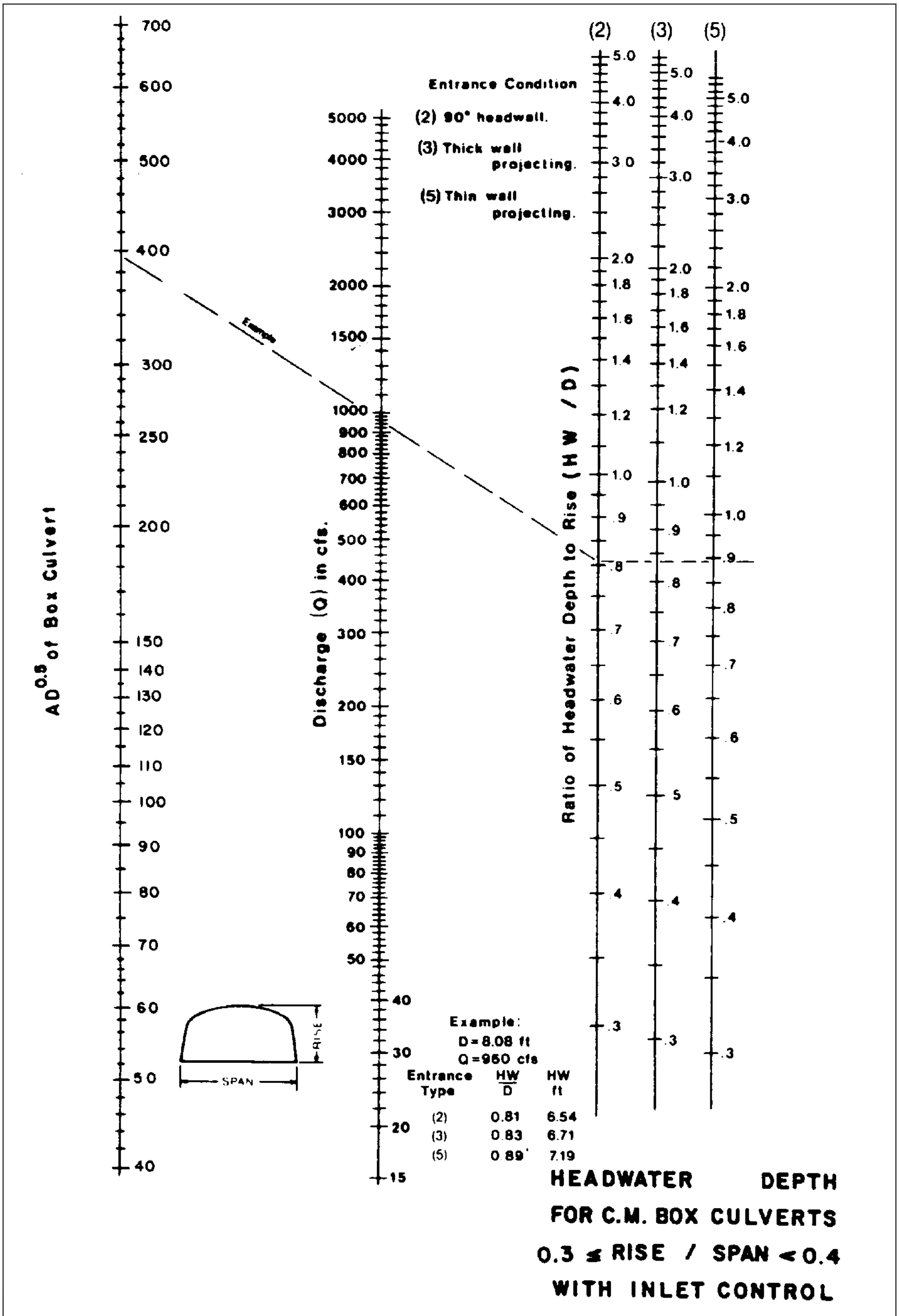
Table 4.19 (continued)

Hydraulic section parameters for structural plate box culverts

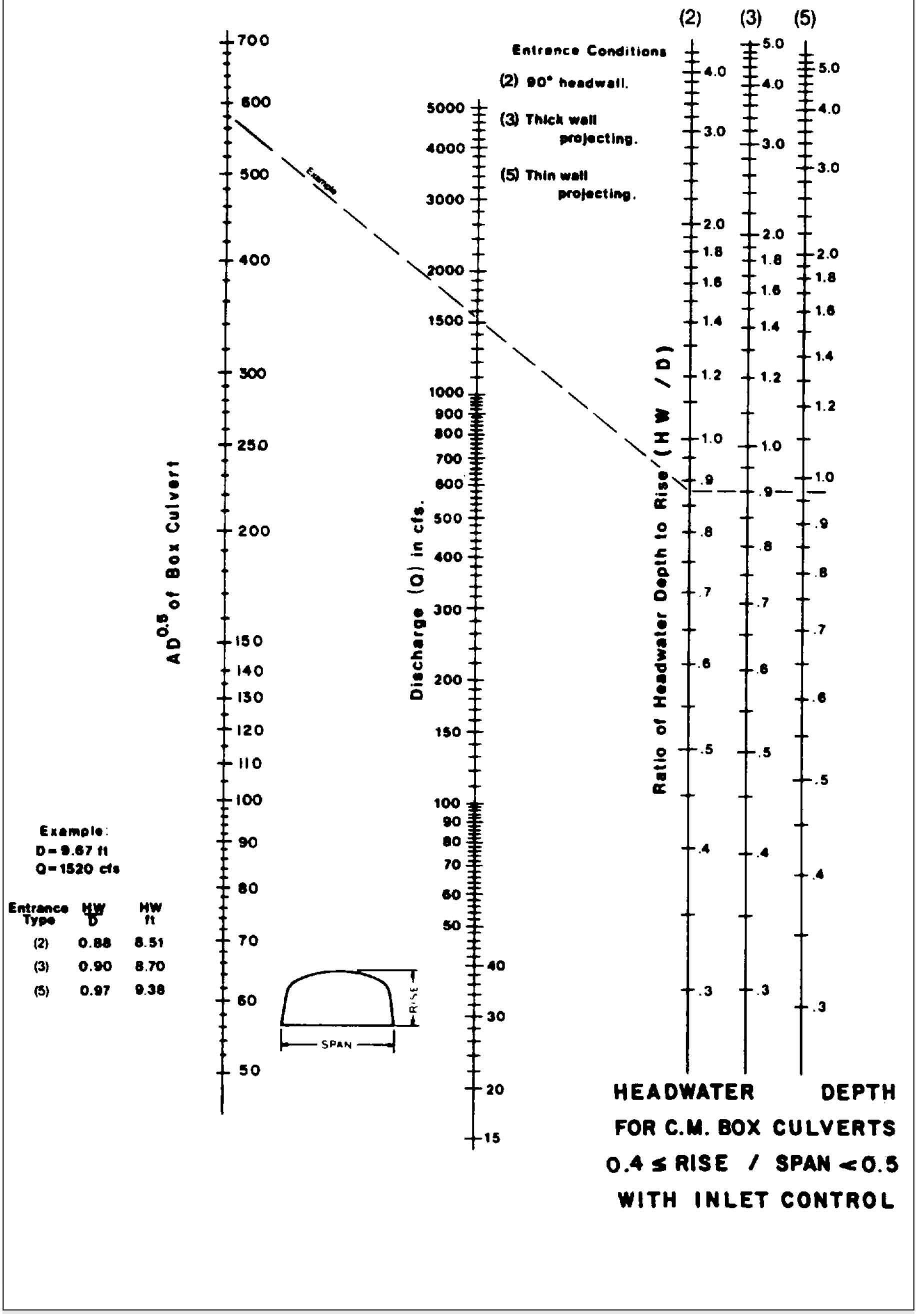
No.	Span ft-in.	Rise ft-in.	Area (sq ft)	WP ft.	AR 2/3	AD 1/2	No.	Span ft-in.	Rise ft-in.	Area (sq ft)	WP ft.	AR 2/3	AD 1/2
55	22-8	5-4	95.0	49.7	146.2	219.4	109	15-8	7-3	98.7	41.1	176.9	265.8
56	14-0	5-5	65.6	64.7	100.3	152.7	110	20-7	7-3	125.0	49.4	232.1	336.6
57	18-0	5-5	82.2	42.2	128.3	191.3	111	22-7	7-3	135.9	52.9	255.0	365.9
58	21-2	5-5	94.1	47.5	148.4	219.0	112	12-10	7-4	76.0	35.0	127.4	205.8
59	11-0	5-7	47.9	29.0	66.9	113.2	113	19-1	7-4	121.5	47.6	226.8	329.0
60	14-10	5-7	70.7	36.8	109.2	167.1	114	24-5	7-4	146.8	56.5	277.6	397.5
61	18-4	5-7	87.9	43.5	140.5	207.7	115	13-4	7-5	80.4	36.1	137.2	219.0
62	11-5	5-8	52.6	29.7	77.0	125.2	116	16-6	7-5	106.5	43.3	194.1	290.0
63	15-5	5-8	75.1	37.6	119.2	178.8	117	16-10	7-6	111.5	44.1	207.0	305.4
64	19-3	5-8	93.3	45.1	151.5	222.1	118	19-10	7-6	129.8	49.5	246.7	355.5
65	22-2	5-8	101.0	49.3	163.0	240.4	119	14-0	7-7	88.4	38.4	154.0	243.4
66	12-0	5-9	57.2	31.1	85.9	137.2	120	14-4	7-8	91.0	38.6	161.3	252.0
67	16-0	5-10	80.7	39.8	129.4	194.9	121	17-6	7-8	119.0	45.6	225.6	329.5
68	19-10	5-10	97.2	46.3	159.4	234.8	121	20-0	7-8	127.1	48.7	240.9	351.9
69	23-9	5-10	108.2	52.8	174.6	261.3	123	21-2	7-8	134.8	50.9	258.1	373.2
70	12-10	5-11	62.3	33.1	94.9	151.5	124	23-6	7-8	150.6	55.6	292.7	417.0
71	16-8	5-11	84.7	40.7	138.0	206.0	125	15-0	7-9	99.4	40.6	180.6	276.7
72	13-2	6-0	66.6	33.8	104.7	163.1	126	25-4	7-9	161.3	59.1	315.1	449.0
73	14-0	6-1	72.1	35.9	114.8	177.8	127	18-3	7-10	125.5	47.2	240.9	351.2
74	17-2	6-1	91.3	42.5	152.1	225.2	128	15-8	7-11	106.8	42.5	197.3	300.5
75	19-2	6-1	97.1	44.7	162.8	239.5	129	18-10	7-11	132.1	48.7	256.8	371.7
76	23-4	6-1	113.7	51.9	191.7	280.4	130	20-10	7-11	143.3	51.7	282.6	403.2
77	20.11	6-2	105.5	48.2	177.9	262.0	131	16-0	8-1	111.8	43.2	210.9	317.9
78	24-10	6-2	119.0	54.3	200.7	295.5	132	19-3	8-1	135.9	49.2	267.4	386.4
79	14-4	6-3	76.6	36.7	125.2	191.5	133	20-11	8-1	141.2	51.3	277.2	401.4
80	18-0	6-3	96.9	43.8	164.4	242.3	134	22-10	8-1	153.7	54.7	306.1	437.0
81	10-10	6-4	53.8	29.6	80.2	135.4	135	16-8	8-2	119.6	44.8	230.2	341.8
82	15-1	6-4	82.5	38.7	136.6	207.6	136	24-6	8-2	166.2	58.1	334.9	475.0
83	18-4	6-4	101.9	44.9	176.0	256.4	137	16-10	8-3	124.4	45.6	243.0	357.3
84	11-9	6-5	61.2	31.7	94.9	155.0	138	20-2	8-3	146.2	51.5	293.2	419.9
85	15-6	6-6	87.1	39.2	148.2	222.1	139	17-1	8-3	125.8	45.7	246.9	363.2
86	19-2	6-6	106.6	46.6	185.2	271.8	140	20-4	8-5	142.6	50.7	284.0	413.7
87	20-2	6-6	109.2	47.5	190.3	278.4	141	21-10	8-5	155.7	53.9	315.9	451.7
88	22-5	6-6	118.4	51.0	207.5	301.9	142	17-8	8-6	133.0	47.5	264.2	387.8
89	24-4	6-6	127.2	55.1	222.3	324.3	143	23-8	8-6	169.3	57.2	349.0	493.6
90	12-5	6-7	66.4	33.0	105.8	170.4	144	18-4	8-7	139.9	48.7	282.5	409.9
91	16-3	6-7	93.5	41.6	160.5	239.9	145	21-2	8-7	160.7	54.1	332.0	470.8
92	13-1	6-8	72.3	35.1	117.1	186.7	146	25-4	8-7	182.4	60.8	379.4	534.4
93	19-8	6-8	113.3	47.6	202.0	292.5	147	18-11	8-9	147.0	50.0	301.7	434.8
94	13-6	6-9	76.9	35.8	128.1	199.8	148	21-3	8-10	157.6	53.4	324.2	468.4
95	16-10	6-9	98.3	42.2	172.8	255.4	149	23-0	8-10	171.6	56.4	360.1	510.0
96	17-5	6-10	105.1	44.3	186.8	274.7	150	19-4	8-11	152.1	51.2	314.3	454.2
97	21-3	6-10	121.7	50.3	219.4	318.1	151	24-6	8-11	185.7	59.9	394.8	554.5
98	14-2	6-11	83.1	31.8	140.6	218.5	152	20-0	9-1	161.6	53.0	339.9	487.0
99	19-7	6-11	111.9	46.8	200.0	294.3	153	21-10	9-3	173.3	55.9	368.4	527.1
100	23-5	6-11	132.1	53.7	240.7	347.4	154	23-10	9-3	188.1	59.2	406.8	572.1
101	14-8	7-0	88.4	38.8	153.1	233.9	155	21-3	9-5	177.4	55.6	384.4	544.4
102	18-1	7-0	111.1	45.2	202.5	293.9	156	25-4	9-5	202.9	62.2	446.1	622.6
103	21-2	7-0	127.4	50.8	235.3	337.1	157	23-2	9-8	189.8	58.4	416.4	590.1
104	25-4	7-0	142.2	57.6	259.8	376.2	158	24-8	9-9	205.3	61.5	458.7	641.0
105	15-5	7-1	94.5	40.6	166.1	251.5	159	24-0	10-1	207.0	60.9	468.0	657.3
106	11-4	7-2	63.0	31.8	99.3	168.7	160	25-5	10-2	222.7	94.0	511.4	710.1
107	18-9	7-2	117.3	46.5	217.2	314.0	161	24-9	10-6	225.0	63.2	524.6	729.1
108	12-3	7-3	71.2	34.4	115.6	191.7							



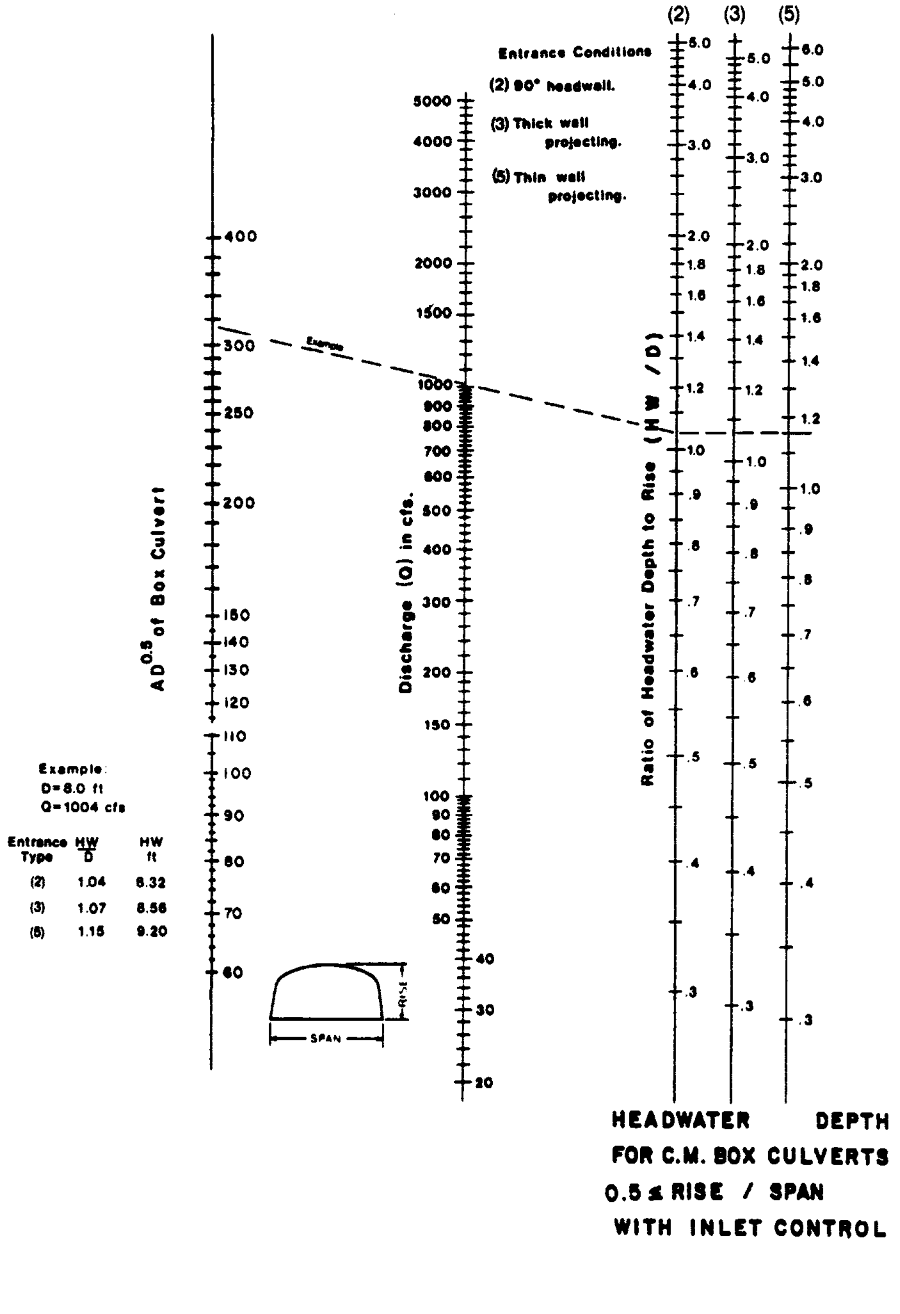
■ **Figure 4.44** Headwater depths for structural plate box culverts, with rise/span < 0.3, under inlet control.



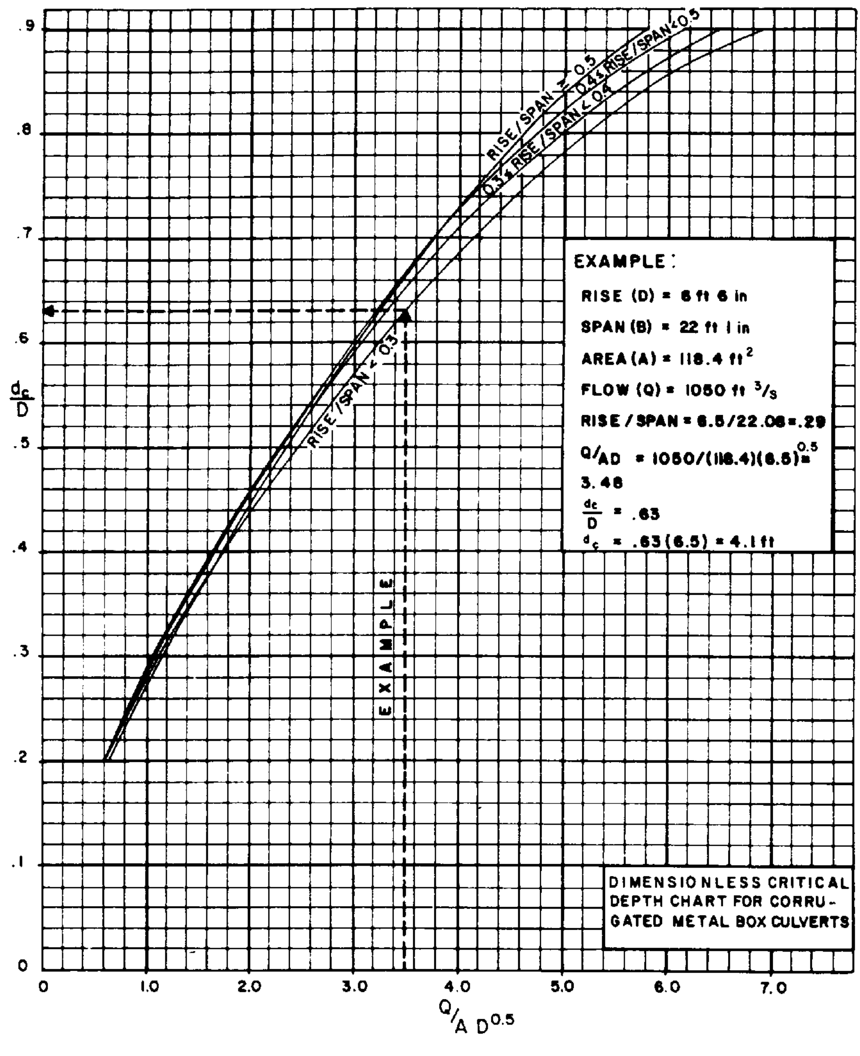
■ **Figure 4.45** Headwater depths for structural plate box culverts, with 0.3 <= rise/span < 0.4, under inlet control.



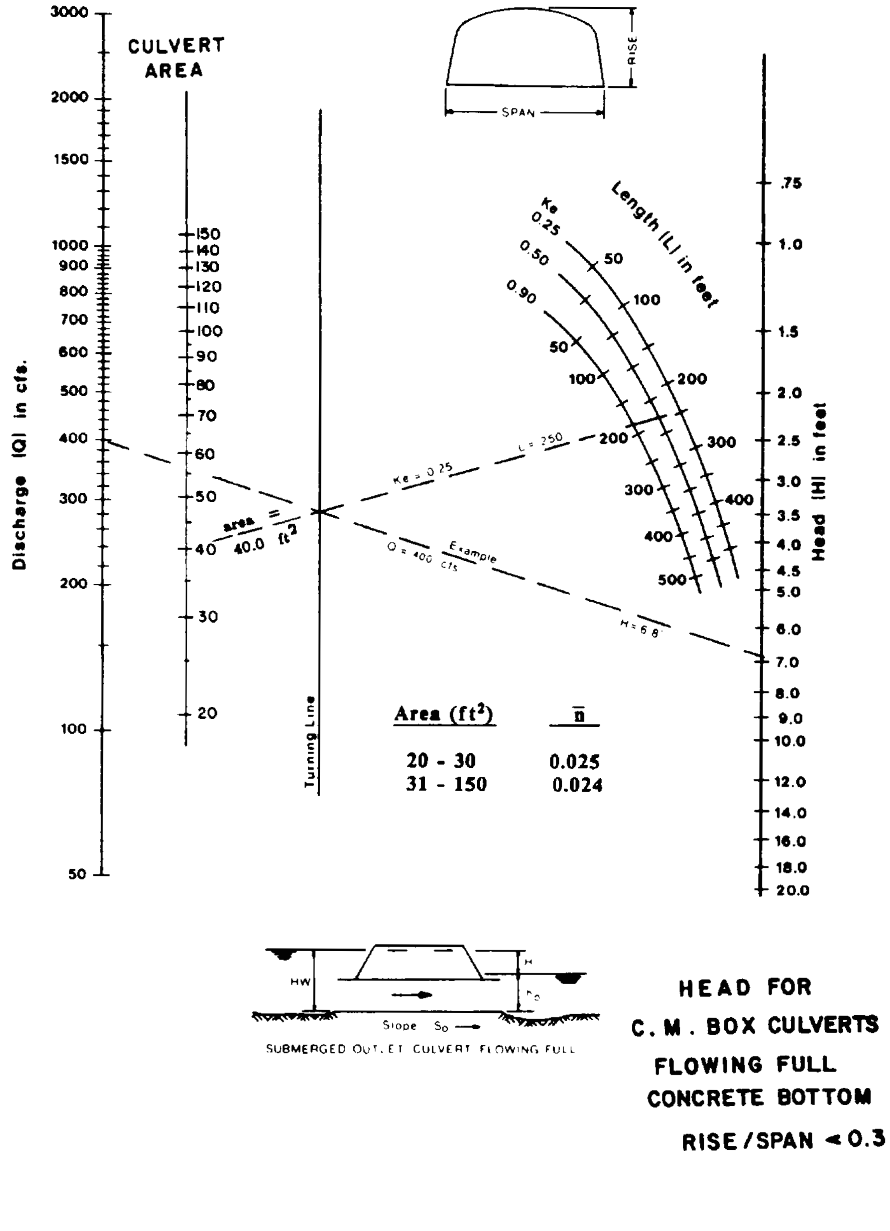
■ **Figure 4.46** Headwater depths for structural plate box culverts, with 0.4 <= rise/span < 0.5, under inlet control.



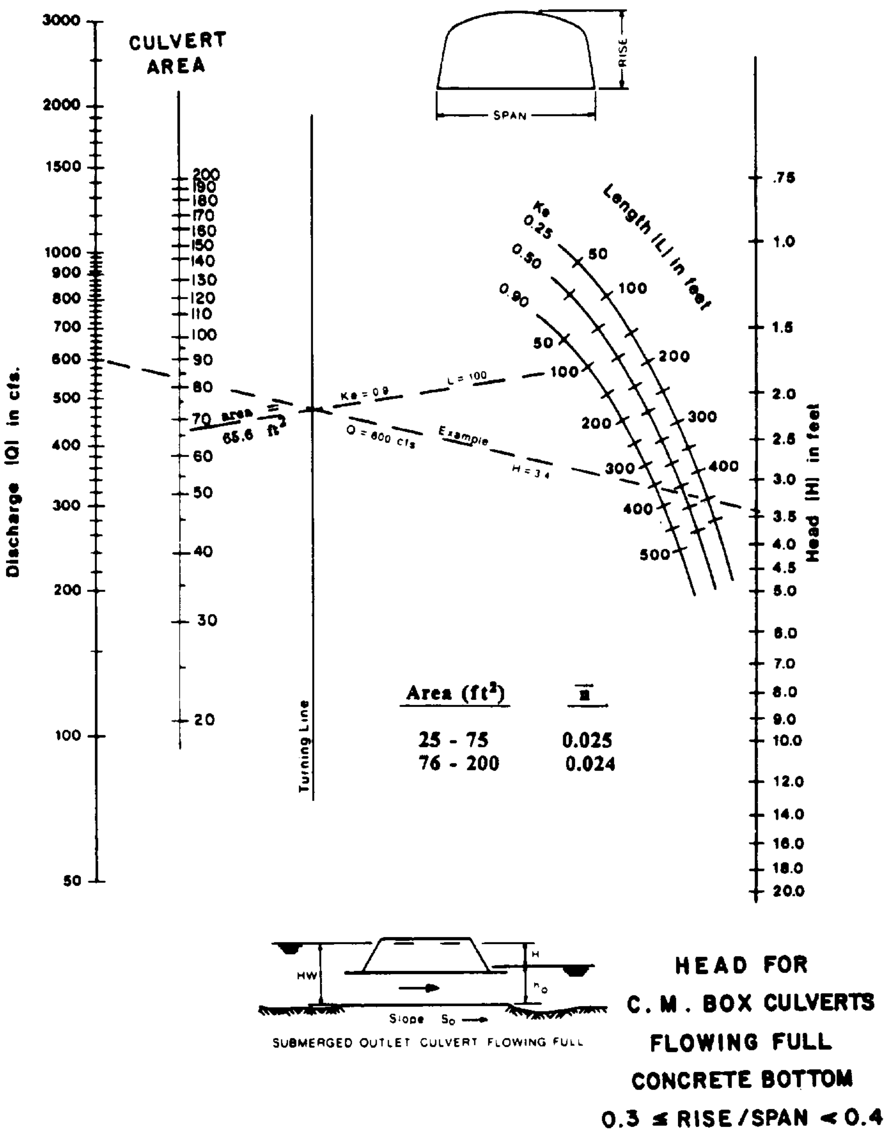
■ **Figure 4.47** Headwater depths for structural plate box culverts, with 0.5 <= rise/span, under inlet control.



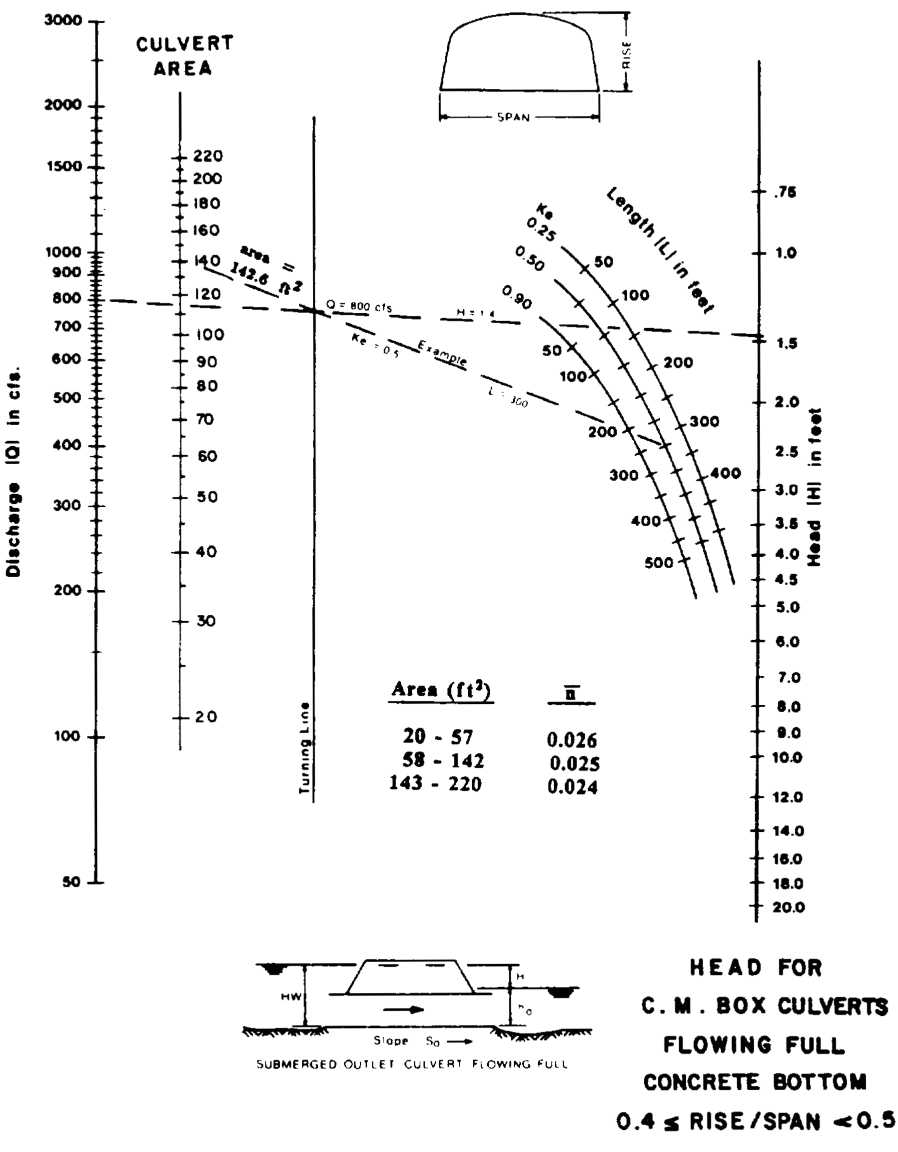
■ **Figure 4.48** Critical depth for corrugated steel structural plate box culverts.



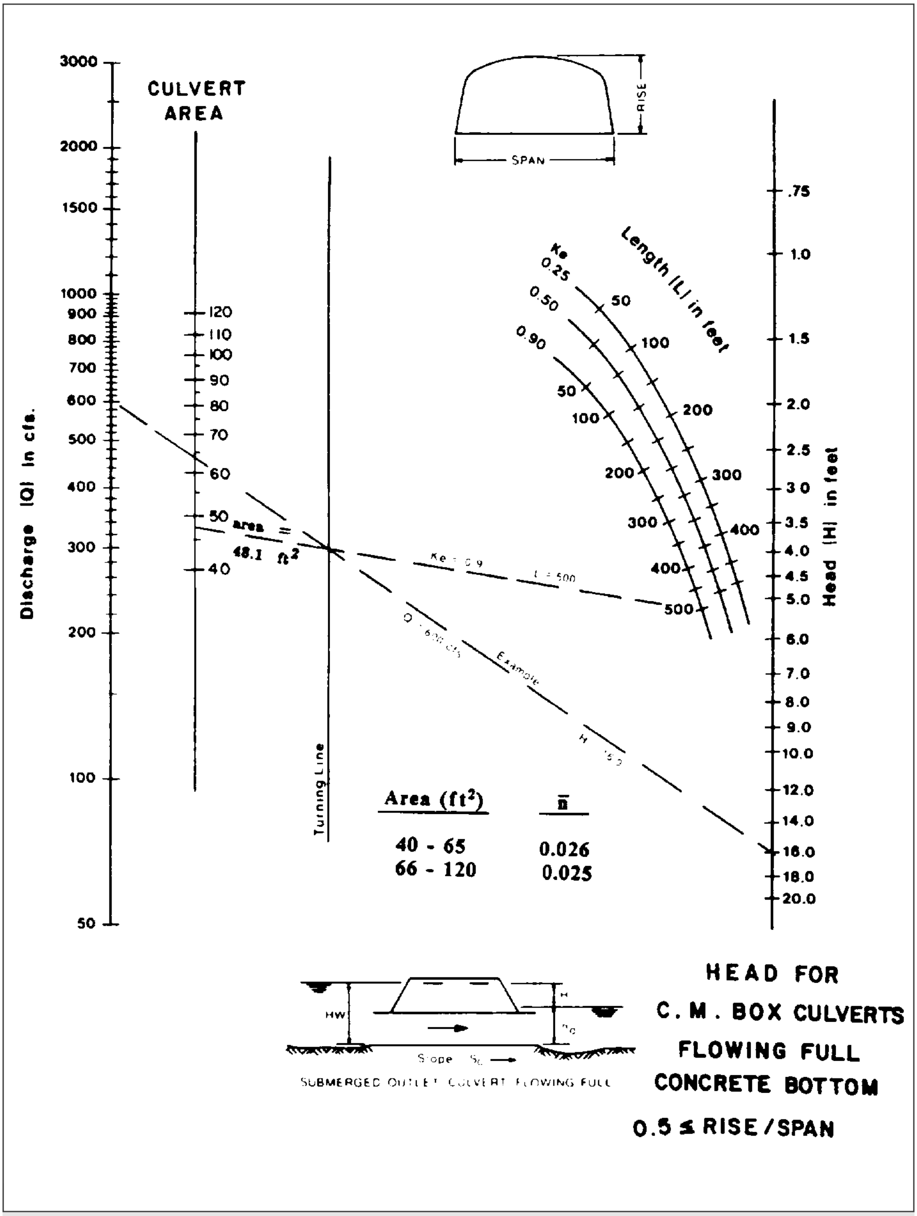
■ **Figure 4.49** Head for 6x2 structural plate box culverts, with concrete invert and rise/span < 0.3, flowing full under outlet control.



■ **Figure 4.50** Head for 6x2 structural plate box culverts, with concrete invert and 0.3 <= rise/span < 0.4, flowing full under outlet control.



■ **Figure 4.51** Head for 6x2 structural plate box culverts, with concrete invert and 0.4 <= rise/span < 0.5, flowing full under outlet control.



■ **Figure 4.52** Head for 6x2 structural plate box culverts, with concrete invert and 0.5 <= rise/span, flowing full under outlet control.

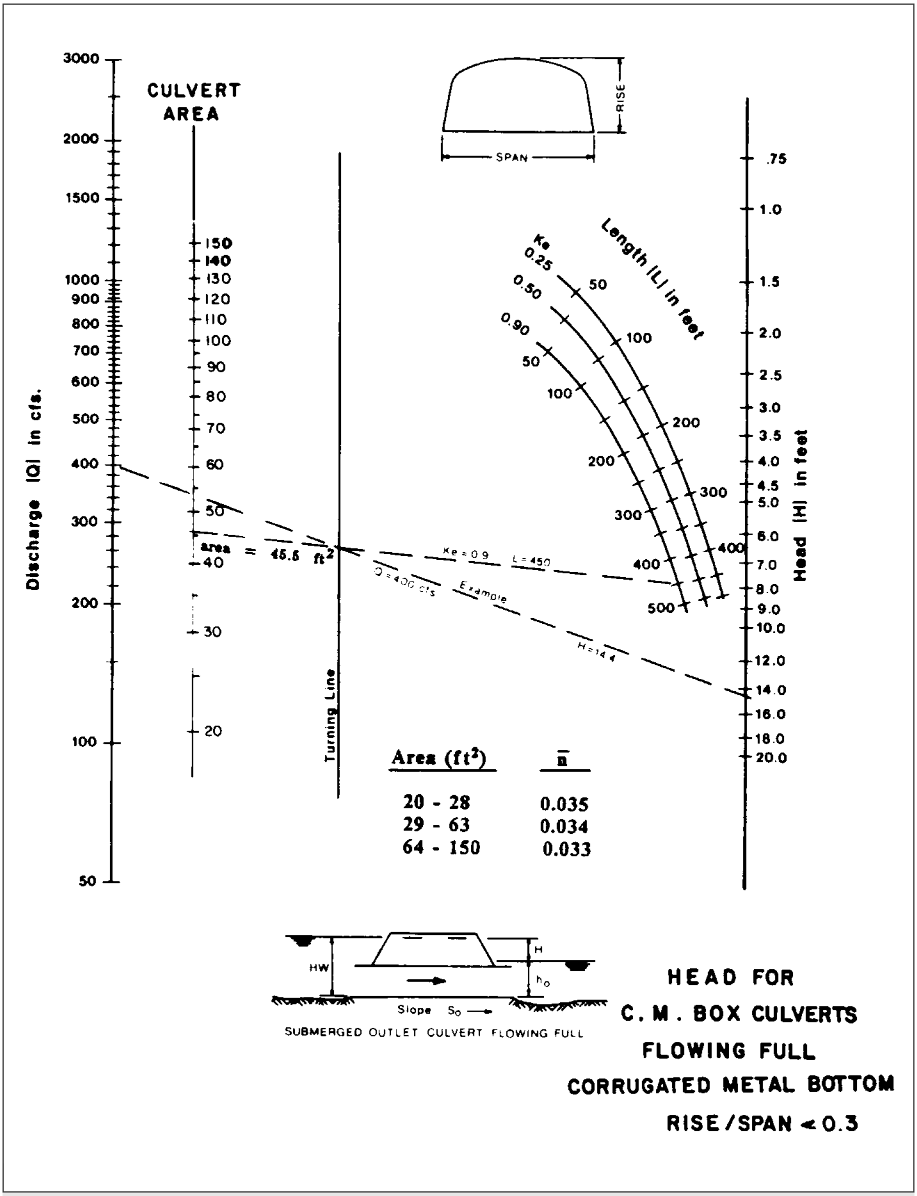


Figure 4.53 Head for 6x2 structural plate box culverts, with corrugated invert and rise/span < 0.3, flowing full under outlet control.

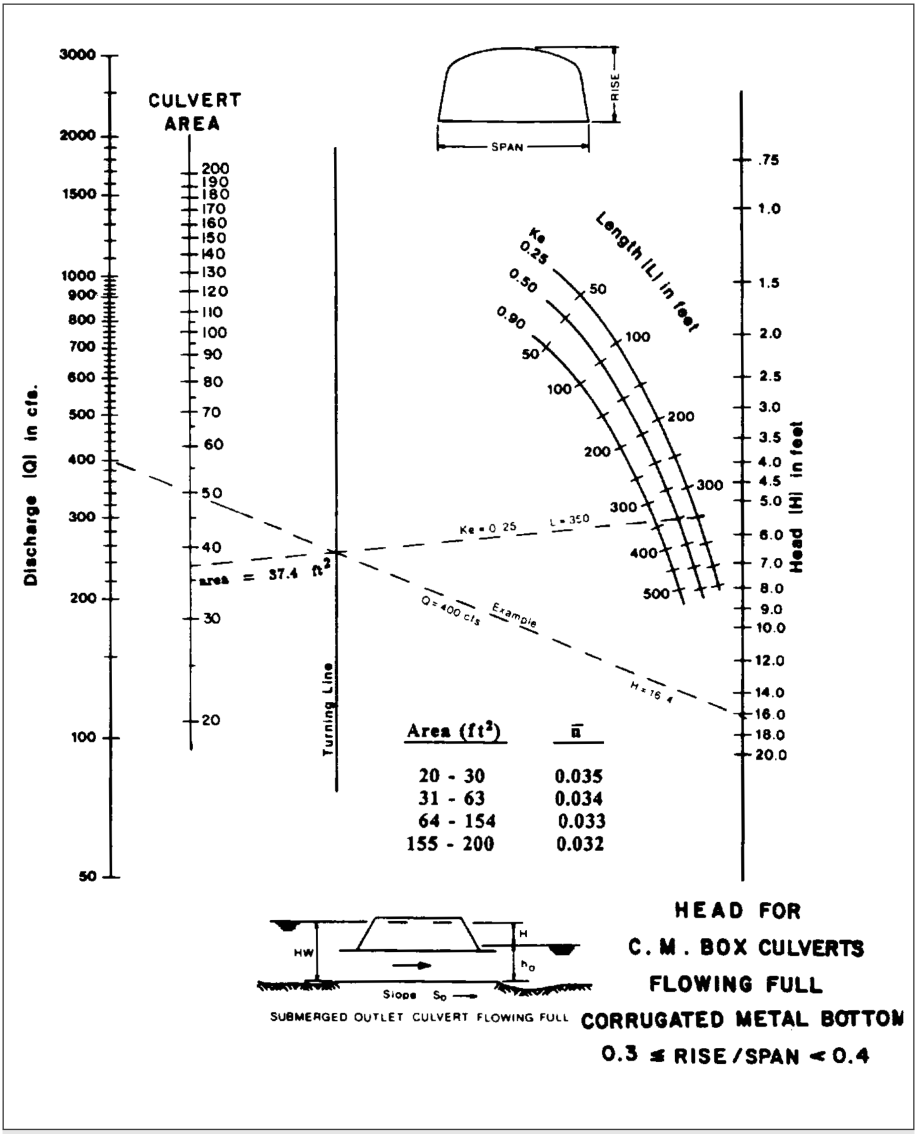
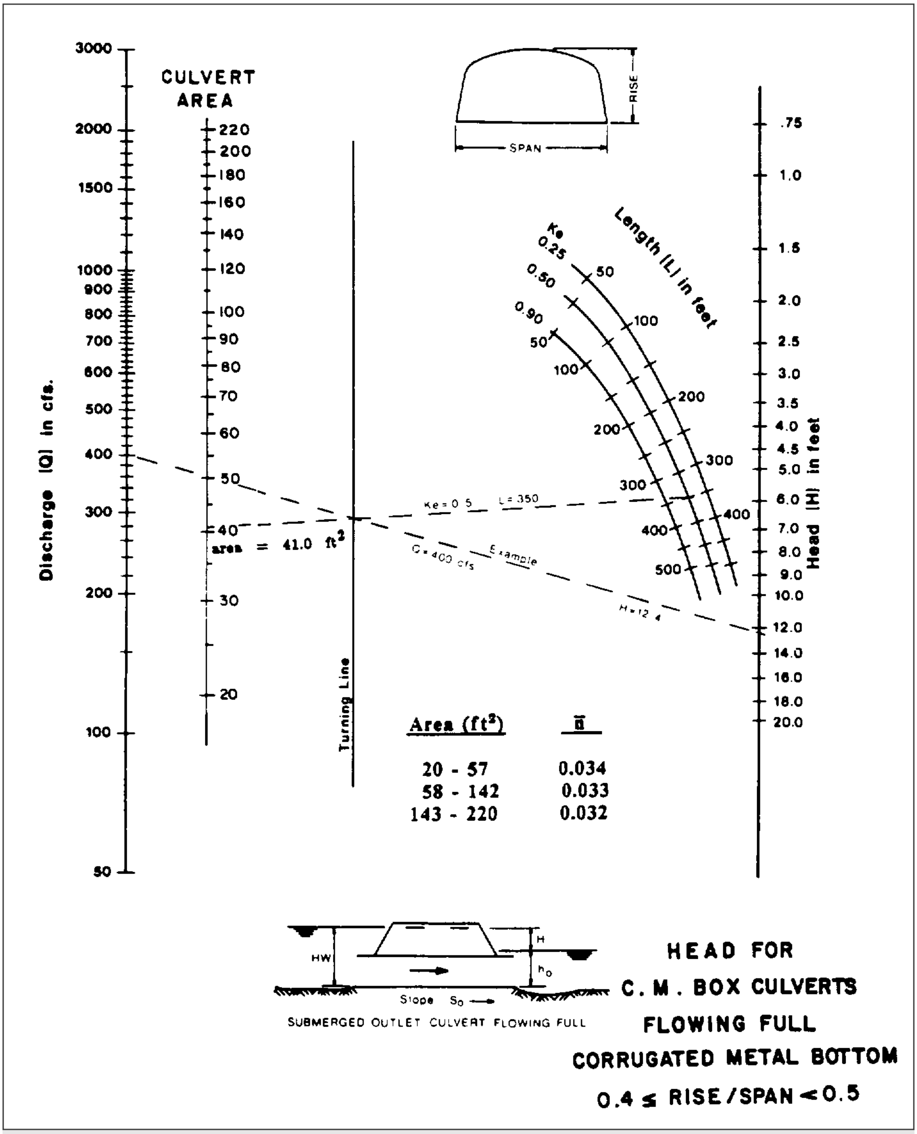
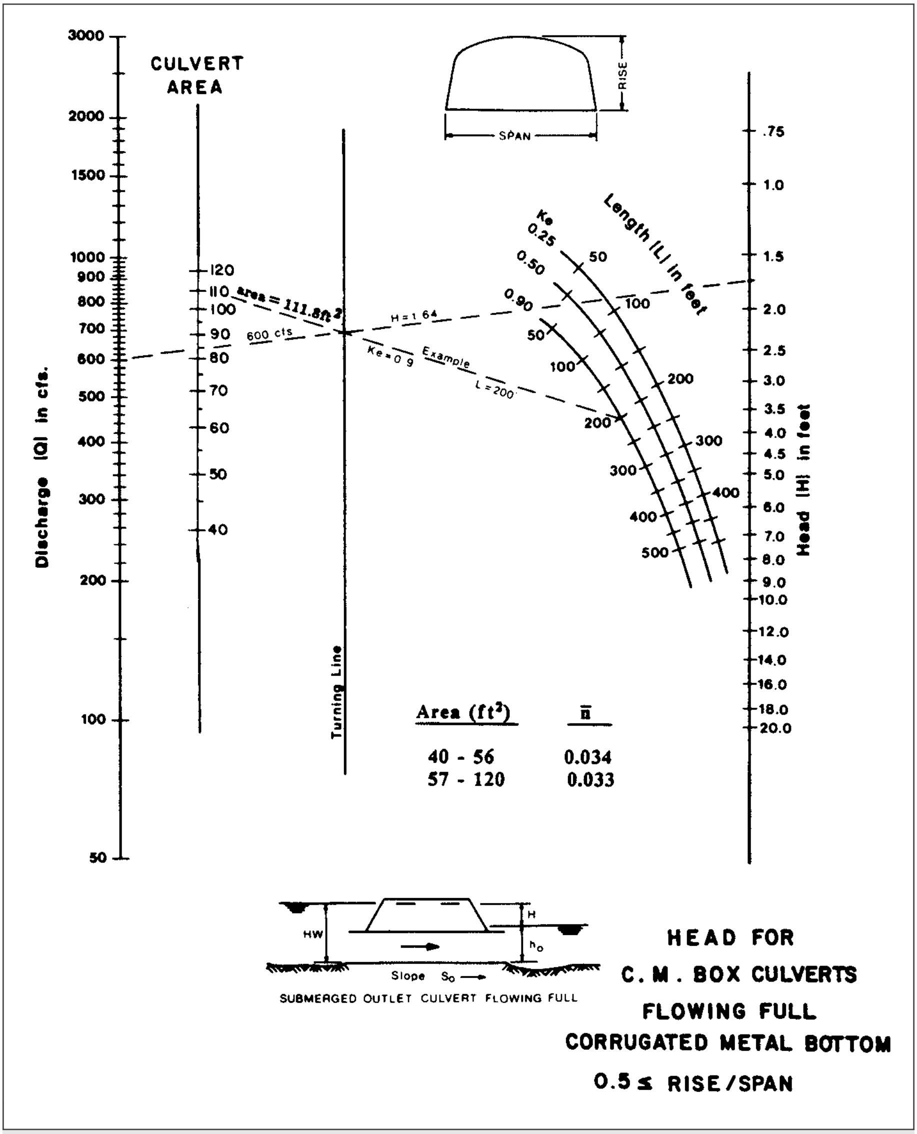


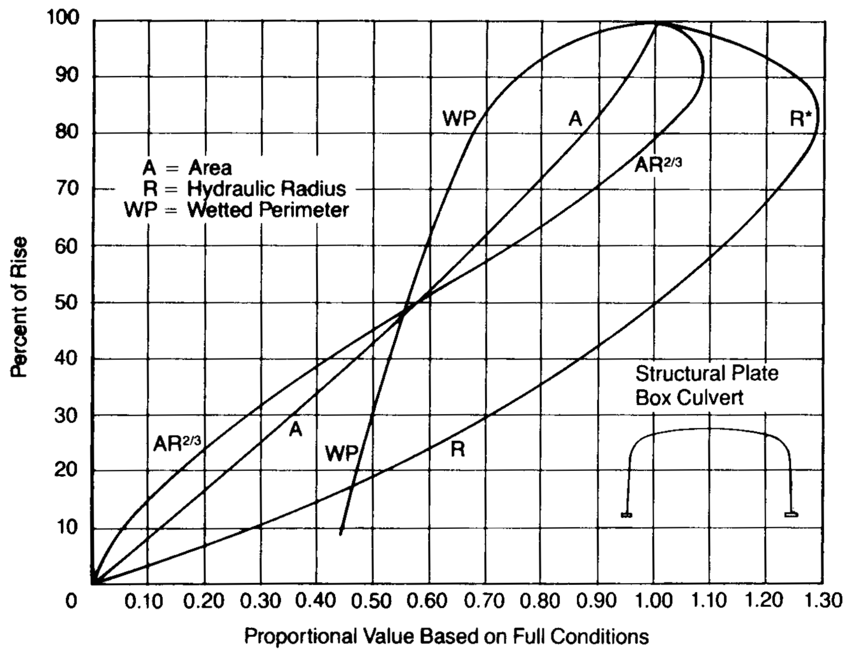
Figure 4.54 Head for 6x2 structural plate box culverts, with corrugated invert and 0.3 <= rise/span < 0.4, flowing full under outlet control.



■ **Figure 4.55** Head for 6x2 structural plate box culverts, with corrugated invert and 0.4 <= rise/span < 0.5, flowing full under outlet control.



■ **Figure 4.56** Head for 6x2 structural plate box culverts, with corrugated invert and 0.5 <= rise/span, flowing full under outlet control.



■ **Figure 4.57** Hydraulic section parameters for structural plate box culverts.

SPECIAL HYDRAULIC CONSIDERATIONS

In addition to flow hydraulics, the drainage designer must consider hydraulic forces and other hydraulic phenomena that may be factors in assuring the integrity of the culvert and embankment.

Uplifting Forces

Uplifting forces on the inlet end of a culvert result from a variety of hydraulic factors that may act on the inlet during high flows. These may include: 1) Vortexes and eddy currents that cause scour, which in turn undermine the inlet and erode the culvert supporting embankment slope; 2) Debris blockage that accentuates the normal flow constriction, creating a larger trapped air space just inside the inlet, resulting in a significant buoyancy force that may lift the inlet; and 3) Sub-atmospheric pressures on the inside of the inlet, combined with flow forces or hydraulic pressures on the outside, that may cause the inward deflection of a skewed or beveled inlet, blocking flow and creating the potential for hydraulic uplift.

Buoyancy type failures can be prevented by structural anchorage of the culvert entrance. This anchorage should be extended into the embankment both below and to the sides of the pipe. Cut-end treatment of the culvert barrel in bevels or skews should have hook bolts embedded in some form of slope protection to protect against bending.

Piping

Piping is a hydraulic phenomena resulting from the submersion of the inlet end of a culvert and high pore pressure in the embankment. Hydrostatic pressure at the inlet will cause the water to seek seepage paths along the outside of the culvert barrel or through the embankment. Piping is the term used to describe the carrying of fill material, usually fines, caused by seepage along the barrel wall. The movement of soil particles through the fill will usually result in voids in the fill. This process has the potential to cause failure of the culvert and/or the embankment. Culvert ends should be sealed where the backfill and embankment material is prone to piping.

Weep Holes

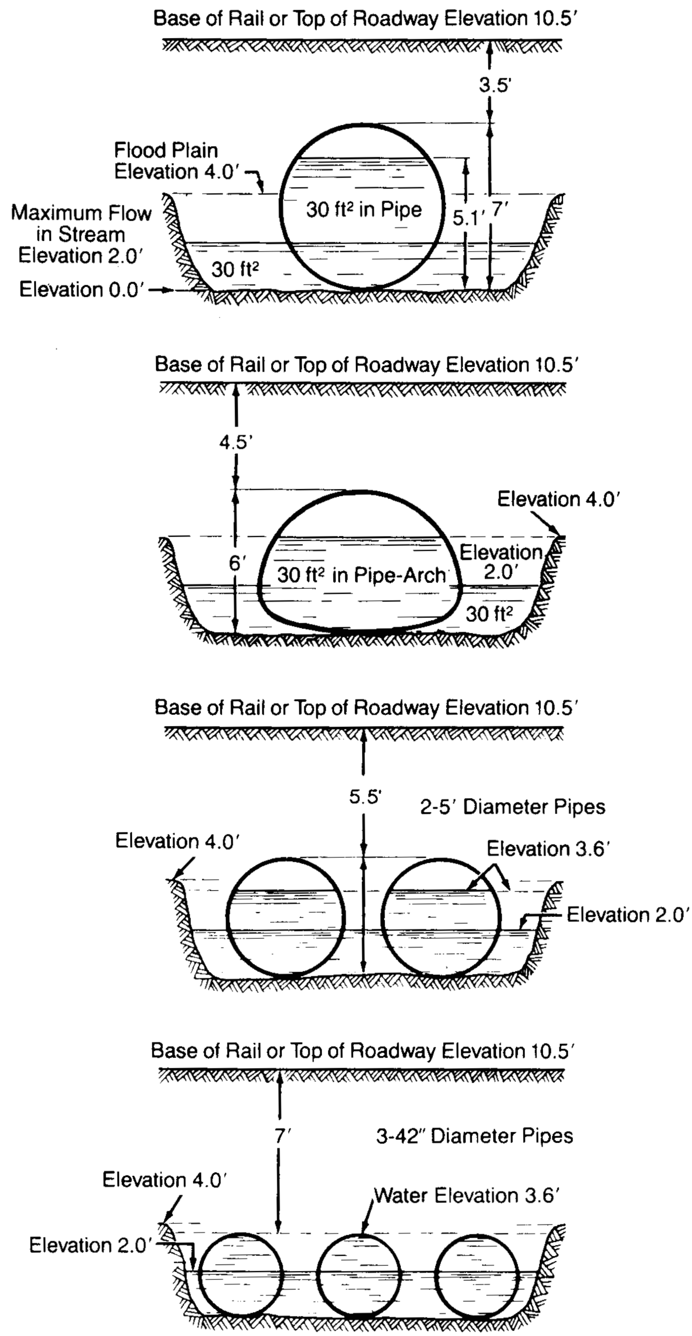
These are perforations in the culvert barrel which are used to relieve pore pressure in the embankment. Generally, weep holes are not required in culvert design. For an installation involving prolonged ponding, there may be merit in considering a separate subdrainage system to relieve pore pressure and control seepage in the embankment.

Anti-Seepage Collars

Vertical cutoff walls may be installed around the culvert barrel at regular intervals to intercept and prevent seepage along the outer wall of the culvert. These may also be referred to as diaphragms. They are most often used in small earth fill dams or levees and are recommended when ponding is expected for an extended time. An example of this is when the highway fill is to be used as a detention dam or temporary reservoir. In such cases, earth fill dam design and construction practices should be considered.

Single vs. Multiple Openings

A single culvert opening is, in general, the most satisfactory because of its greater ability to pass floating debris and driftwood. However, in many cases, the design requires that the waterway be wide in order to get the water through quickly without ponding and flooding of the land upstream. In such cases, the solution may consist of using either an arch, a pipe arch or a battery of two or more openings. See Figure 4.58.



■ **Figure 4.58** Comparison of single and multiple opening installations, and round and pipe arch installations.

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Pipe inspection.

CHAPTER five

Hydraulic Design of Storm Sewers

INTRODUCTION

Storm sewers may be designed as either open channels, where there is a free water surface, or as pipes, where the flow is under surcharged conditions (pressure flow). When the storm sewer system is to be designed as a pressure pipe, the hydraulic grade line can not exceed the floor level of adjacent basements, or catch basin grate opening elevations, if surcharge conditions would create unacceptable flooding or structural damage.

Regardless of whether the sewer system is to be designed as an open channel or pressure system, a thorough hydraulic analysis should be performed to assure that the system operates efficiently. A simplistic approach to the design of storm sewers, with the design and sizing of pipes and appurtenances derived from nomographs or basic hydraulic flow equations, has too often been used.

As a result, excessive surcharging has been experienced in many instances due to improper design of the hydraulic structures. This in turn has led to flood damage, both surface and structural, when service connections have been made to the storm sewer. Overloading of the sewer system may occur in upper reaches, while lower segments may be flowing well below capacity, because of the inability of the upper reaches to transport the flow. Conversely, downstream surcharging can create problems while upper segments are flowing well below capacity.

An efficient, cost-effective storm drain system cannot be designed without a complete and proper hydraulic analysis.

This chapter outlines the basic hydraulic principles for open channel and pipe flow. Losses (friction and form) within the sewer system, the hydraulics of storm water inlets, and the hydraulics of subdrains are also discussed. Manual calculations for designing a storm drainage system are presented and an overview of several commonly used computer programs that may be used to design sewer systems is also given.

HYDRAULICS OF STORM SEWERS

Classification of Channel Flow

Channel flow is distinguished from closed-conduit or pipe flow by the fact that the cross-section of flow is not dependent solely on the geometry of the pipe. It also depends on the free surface (or depth), which varies with respect to space and time and is a function of discharge. As a result, various categories of flow can be identified:

STEADY flow exhibits characteristics at a point that are constant with respect to time. Flow subject to very slow change may be assumed to be steady with little error.

UNSTEADY flow results when a time-dependent boundary condition (tide, flood-wave or gate movement, for example) causes a change in flow and/or depth to be propagated through the system.

UNIFORM flow occurs when the velocity is the same in magnitude and direction at every point in the pipe. Uniform flow is also usually assumed to occur when the velocity at corresponding points in the cross-section is the same along the length of the channel. Note that uniform flow is possible only if:

- flow is steady, or nearly so;
- the channel is prismatic (i.e., has the same cross-sectional shape at all sections);
- depth is constant along the length of the channel; and
- the slope is equal to the energy gradient.

NON-UNIFORM or VARIED flow occurs when any of the requirements for uniform flow are not satisfied. Varied flow may be further sub-classified depending on the abruptness of the variation:

GRADUALLY VARIED flow occurs when depth changes occur over long distances such as the flow profiles or backwater profiles that occur between distinct reaches of uniform flow.

RAPIDLY VARIED flow occurs in the vicinity of transitions caused by relatively abrupt changes in channel geometry or where a hydraulic jump occurs.

In the design of sewer systems, the flow, except where backwater or surcharging may occur, is generally assumed to be steady and uniform.

Figure 5.1 illustrates various typical occurrences of the different classes of flow.

Laws of Conservation

Fluid mechanics is based on the law of conservation applied to the mass, energy and momentum of a fluid in motion. Full details can be found in any text on the subject. At this point, it is sufficient to note that:

a) Conservation of mass reduces to a simple statement of continuity for fluids in which the density is essentially constant.
b) Conservation of energy is usually stated as the Bernoulli equation, which is discussed below.
c) Conservation of momentum is significant in transitions where there are local and significant losses of energy, such as across a hydraulic jump.

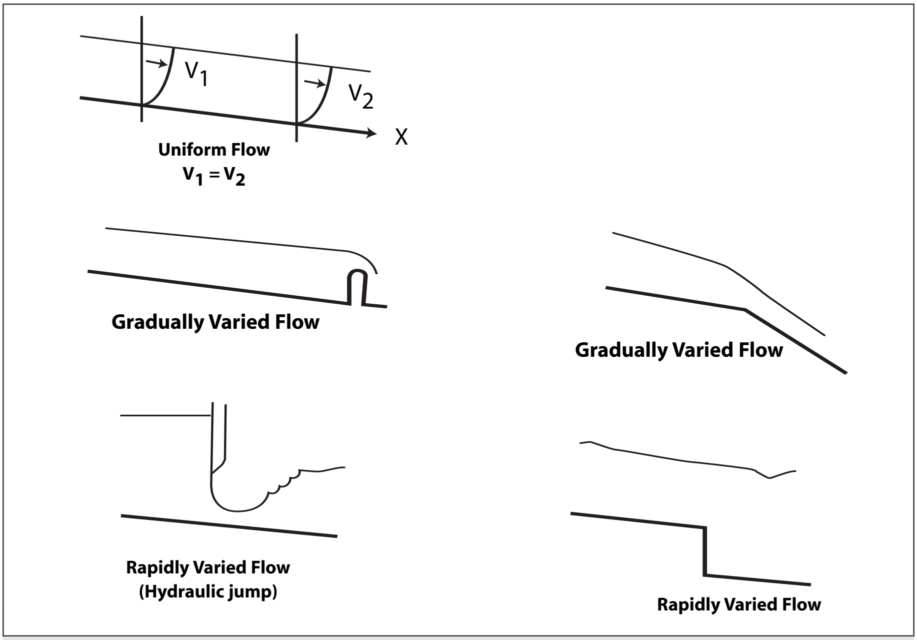


Figure 5.1 Different classes of open channel flow.

Bernoulli Equation

The law of conservation of energy as expressed by the Bernoulli Equation is the basic principle most often used in hydraulics. This equation may be applied to any pipe with a constant discharge. All the friction flow formulae, such as those by Manning, Cutter, and Hazen-Williams, have been developed to express the rate of energy dissipation as it applies to the Bernoulli Equation. The theorem states that the energy head at any cross-section must equal that in any other downstream section plus the intervening losses.

In open channels, the flow is primarily controlled by the gravitational action on moving fluid, which overcomes the hydraulic energy losses. The Bernoulli Equation, defining the hydraulic principles involved in open channel flow (Figure 5.2), is as follows:

$$H = y + \frac{V^2}{2g} + Z + h_f$$

where:
- H = Total energy head, ft
- y = Water depth, ft
- V = Mean velocity, ft/s
- g = Gravitational constant = 32.2 ft/s^2
- Z = Height above datum, ft
- h_f = Head loss, ft

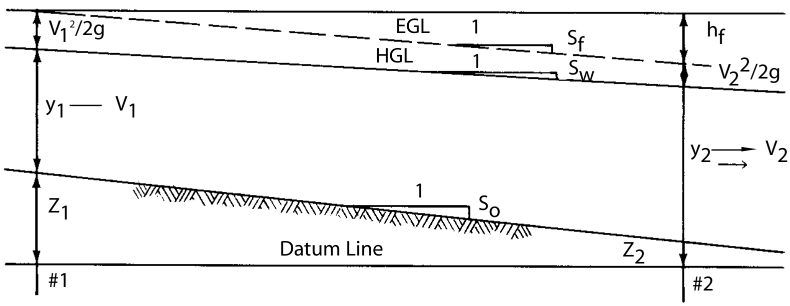


Figure 5.2 Energy in open channel flow.

Other terms shown in Figure 5.2 are defined as follows:

EGL = Energy Grade Line
HGL = Hydraulic Grade Line
S_o = Slope of channel bottom
S_f = Slope of the EGL
S_w = Slope of the HGL
$V^2/2g$ = Velocity head

In Figure 5.2, the total energy at point 1 is equal to the total energy at point 2, and thus:

$$y_1 + Z_1 + \frac{V_1^2}{2g} = y_2 + Z_2 + \frac{V_2^2}{2g} + h_f$$

For pressure or closed pipe flow, as shown in Figure 5.3, the Bernoulli Equation can be written as follows:

$$\frac{V_1^2}{2g} + \frac{P_1}{\gamma} + Z_1 = \frac{V_2^2}{2g} + \frac{P_1}{\gamma} + Z_2 + h_f$$

where: P = Pressure at a given point, lb/in^2
γ = Specific weight of fluid, lb/in^3

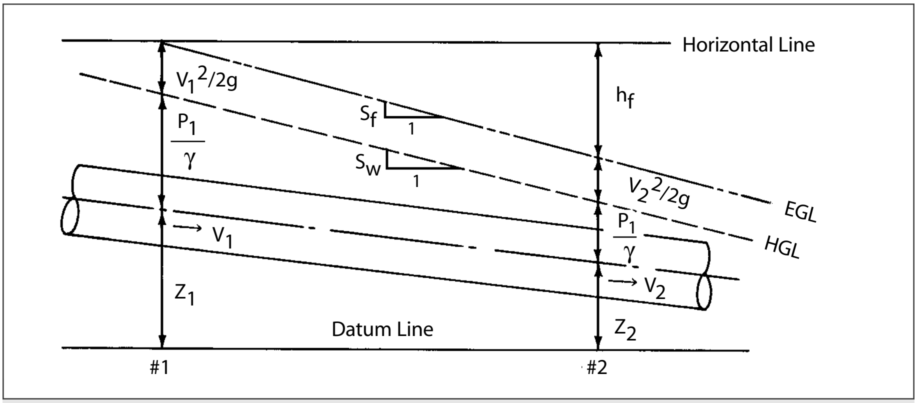


Figure 5.3 Energy in closed pipe flow.

Specific Energy

An understanding of open channel flow is aided by the concept of Specific Energy, E, which is simply the total energy when the channel bottom is taken to be the datum. This may be expressed as:

$$E = y + \frac{V^2}{2g} = y + \frac{Q^2}{2gA^2}$$

Figure 5.4 shows a plot of depth of flow, y, and specific energy, E, as a function of depth of flow for a known cross-sectional shape and constant discharge Q. The minimum value of E occurs at a depth of flow termed the critical depth, y_{cr}. The critical depth is defined by setting dE/dy = 0, from which it can be shown that:

$$\frac{Q^2\ T}{gA^3} = 1$$

where the surface width, T, and cross-sectional area, A, are functions of the depth, y. Table 5.1 lists the cross-sectional areas for a number of standard pipe shapes and sizes. Tables 5.2 through 5.4 provide the means to determine the flow area, hydraulic radius and top width for round pipes flowing partly full. Tables 5.5 through 5.7 provide the same for pipe arches. Note that pipe arch values are approximate since actual pipe arch shapes vary in cross-sectional geometry. An explanation of the use of these tables is provided under Table 5.4

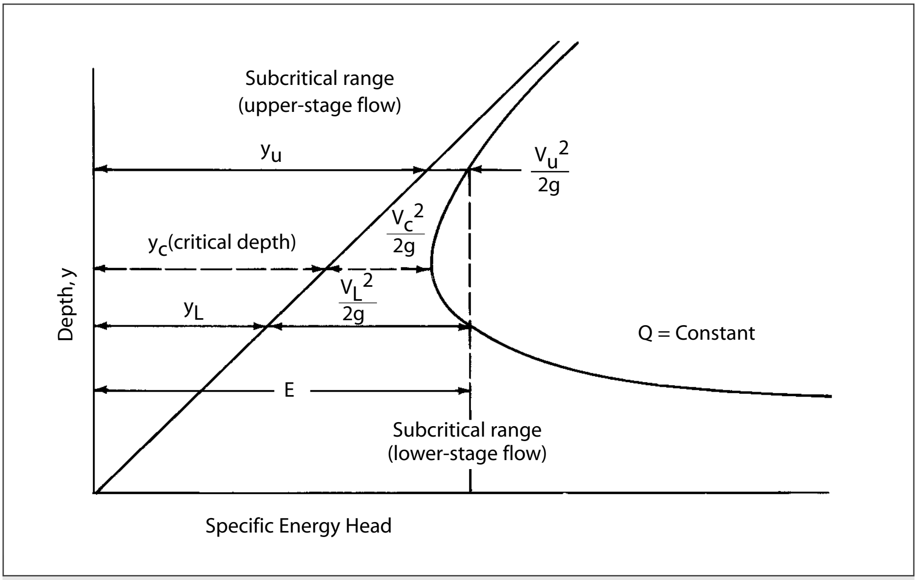


Figure 5.4 Specific energy as a function of depth.

The velocity corresponding to the critical depth is called the critical velocity and is given by:

$$\frac{V^2_{cr}\,T}{gA} = 1 \quad \text{or} \quad V_{cr} = \left(\frac{gA}{T}\right)^{\frac{1}{2}}$$

The critical velocity, and hence the critical depth, is unique to a known cross-sectional shape and constant discharge, Q.

For the special case of rectangular cross-sections, A = B•y and T = B, where B is the channel width. In this case, the above equation for critical depth reduces to:

$$\frac{Q^2}{g \bullet B^2 \bullet y^3} = 1$$

from which the critical depth is found as:

$$y_{cr} = \left(\frac{Q^2}{g \bullet B^2}\right)^{\frac{1}{3}}$$

and the corresponding critical velocity is $V_{cr} = (g \bullet y)^{\frac{1}{2}}$.

Table 5.1

Waterway areas for standard sizes of corrugated steel conduits

Round Pipe		Pipe Arch (1/2 in. deep corrugation)		Structural Plate Pipe Arch	
Diameter	Area	Size	Area	Size	Area
				18-inch Corner Radius	
(in.)	(ft^2)	(in.)	(ft^2)	(ft-in.)	(ft^2)
12	0.785	17 x 13	1.1	6-1 x 4-7	22
15	1.227	21 x 15	1.6	6-4 x 4-9	24
18	1.767	24 x 18	2.2	6-9 x 4-11	26
21	2.405	28 x 20	2.9	7-0 x 5-1	28
24	3.142	35 x 24	4.5	7-3 x 5-3	31
30	4.909	42 x 29	6.5	7-8 x 5-5	33
36	7.069	49 x 33	8.9	7-11 x 5-7	35
42	9.621	57 x 38	11.6	8-2 x 5-9	38
48	12.566	64 x 43	14.7	8-7 x 5-11	40
54	15.904	71 x 47	18.1	8-10 x 6-1	43
60	19.635	77 x 52	21.9	9-4 x 6-3	46
66	23.758	83 x 57	26.0	9-6 x 6-5	49
72	28.27			9-9 x 6-7	52
78	33.18	Pipe Arch (1 in. deep corrugation)		10-3 x 6-9	55
84	38.49			10-8 x 6-11	58
90	44.18	Size	Area	10-11 x 7-1	61
96	50.27			11-5 x 7.3	64
108	63.62	60 x 46	15.6	11-7 x 7.5	67
114	70.88	66 x 51	19.3	11-10 x 7-7	71
120	78.54	73 x 55	23.2	12-4 x 7-9	74
132	95.03	81 x 59	27.4	12-6 x 7-11	78
138	103.87	87 x 63	32.1	12-8 x 8-1	81
144	113.10	95 x 67	37.0	12-10 x 8-4	85
150	122.7	103 x 71	42.4	13-5 x 8-5	89
156	132.7	112 x 75	48.0	13-11 x 8-7	93
162	143.1	117 x 79	54.2	14-1 x 8-9	97
168	153.9	128 x 83	60.5	14-3 x 8-11	101
174	165.1	137 x 87	67.4	14-10 x 9-1	105
180	176.7	142 x 91	74.5	15-4 x 9-3	109
186	188.7	Structural Plate Arch		15-6 x 9-5	113
192	201.1	(2 in. deep corrugation)		15-8 x 9-7	118
198	213.8	Size	Area	15-10 x 9-10	122
204	227.0			16-5 x 9-11	126
210	240.5	6.0 x 3-2	15	16-7 x 10-1	131
216	254.0	7.0 x 3-8	20	31 in. Corner Radius	
222	268.8	8.0 x 4-2	26		
228	283.5	9.0 x 4-8.5	33	13-3 x 9-4	97
234	298.6	10.0 x 5-3	41	13-6 x 102	102
240	314.2	11.0 x 5-9	50	14-0 x 9-8	105
246	330.1	12.0 x 6-3	59	14-2 x 9-10	109
252	346.4	13.0 x 6-9	70	14-5 x 10-0	114
258	363.1	14.0 x 7-3	80	14-11 x 10-2	118
264	380.1	15.0 x 7-9	92	15-4 x 10-4	123
270	397.6	16.0 x 8-3	105	15-7 x 10-6	127
276	415.5	17.0 x 8-10	119	15-10 x 10-8	132
282	433.7	18.0 x 8-11	126	16-3 x 10-10	137
288	452.4	19.0 x 9-5.5	140	16-6 x 11-0	142
294	471.4	20.0 x 10-0	157	17-0 x 11-2	146
300	490.9	21.0 x 10-6	172	17-2 x 11-4	151
		22.0 x 11-0	190	17-5 x 11-6	157
		23.0 x 11-6	208	17-11 x 11-8	161
		24.0 x 12-0	226	18-1 x 11-10	167
		25.0 x 12-6	247	18-7 x 12-0	172
				18-0 x 12-2	177
				19-3 x 12-4	182
				19-6 x 12-6	188
				19-8 x 12-8	194
				19-11 x 12-10	200
				20-5 x 13-0	205
				20-7 x 13-2	211

Hydraulic Properties of Circular Conduits Flowing Part Full*

D = Diameter
y = Depth of flow
A = Area of flow
R = Hydraulic radius
T = Top width

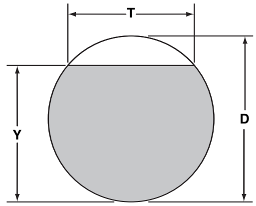


Table 5.2

Determination of area — Values of $\frac{A}{D^2}$

$\frac{y}{D}$	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
.0	.000	.001	.004	.007	.011	.015	.019	.024	.029	.035
.1	.041	.047	.053	.060	.067	.074	.081	.089	.096	.104
.2	.112	.120	.128	.136	.145	.154	.162	.171	.180	.189
.3	.198	.207	.217	.226	.236	.245	.255	.264	.274	.284
.4	.293	.303	.313	.323	.333	.343	.353	.363	.373	.383
.5	.393	.403	.413	.423	.433	.443	.453	.462	.472	.482
.6	.492	.502	.512	.521	.531	.540	.550	.559	.569	.578
.7	.587	.596	.605	.614	.623	.632	.640	.649	.657	.666
.8	.674	.681	.689	.697	.704	.712	.719	.725	.732	.738
.9	.745	.750	.756	.761	.766	.771	.775	.779	.782	.784
1.0	.785									

Table 5.3

Determination of hydraulic radius — Values of $\frac{R}{D}$

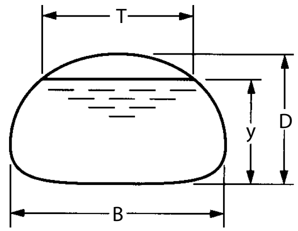
$\frac{y}{D}$	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
.0	.000	.007	.013	.020	.026	.033	.039	.045	.051	.057
.1	.063	.070	.075	.081	.087	.093	.099	.104	.110	.115
.2	.121	.126	.131	.136	.142	.147	.152	.157	.161	.166
.3	.171	.176	.180	.185	.189	.193	.198	.202	.206	.210
.4	.214	.218	.222	.226	.229	.233	.236	.240	.243	.247
.5	.250	.253	.256	.259	.262	.265	.268	.270	.273	.275
.6	.278	.280	.282	.284	.286	.288	.290	.292	.293	.295
.7	.296	.298	.299	.300	.301	.302	.302	.303	.304	.304
.8	.304	.304	.304	.304	.304	.303	.303	.302	.301	.299
.9	.298	.296	.294	.292	.289	.286	.283	.279	.274	.267
1.0	.250									

Table 5.4

Determination of top width — Values of $\frac{T}{D}$

$\frac{y}{D}$	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
.0	.000	.199	.280	.341	.392	.436	.457	.510	.543	.572
.1	.600	.626	.650	.673	.694	.714	.733	.751	.768	.785
.2	.800	.815	.828	.842	.854	.866	.877	.888	.898	.908
.3	.917	.925	.933	.940	.947	.954	.960	.966	.971	.975
.4	.980	.984	.987	.990	.993	.995	.997	.998	.999	1.000
.5	1.000	1.000	.999	.998	.997	.995	.993	.990	.987	.984
.6	.980	.975	.971	.966	.960	.954	.947	.940	.933	.925
.7	.917	.908	.898	.888	.877	.866	.854	.842	.828	.815
.8	.800	.785	.768	.751	.733	.714	.694	.673	.650	.626
.9	.600	.572	.543	.510	.475	.436	.392	.341	.280	.199
1.0	.000									

*Example: y = 36 in.; D = 48 in.; From tables, $\frac{A}{D^2} = 0.632$, $\frac{R}{D} = 0.302$, $\frac{T}{D} = 0.866$



Hydraulic Properties of Pipe Arch Conduits Flowing Part Full

y = Depth of flow
D = Rise of conduit
B = Span of conduit

A = Area of flow
R = Hydraulic radius
T = Top width of flow

Table 5.5

Determination of area — Values of $\frac{A}{BD}$

$\frac{y}{D}$	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
.1		.072	.081	.090	.100	.109	.119	.128	.138	.148
.2	.157	.167	.177	.187	.197	.207	.217	.227	.237	.247
.3	.257	.267	.277	.287	.297	.307	.316	.326	.336	.346
.4	.356	.365	.375	.385	.394	.404	.413	.423	.432	.442
.5	.451	.460	.470	.479	.488	.497	.506	.515	.524	.533
.6	.541	.550	.559	.567	.576	.584	.592	.600	.608	.616
.7	.624	.632	.640	.647	.655	.662	.670	.677	.684	.690
.8	.697	.704	.710	.716	.722	.728	.734	.740	.745	.750
.9	.755	.760	.764	.769	.772	.776	.780	.783	.785	.787
1.0	.788									

Table 5.6

Determination of hydraulic radius — Values of $\frac{R}{D}$

$\frac{y}{D}$	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
.1		.078	.086	.094	.102	.110	.118	.126	.133	.141
.2	.148	.156	.163	.170	.177	.184	.191	.197	.204	.210
.3	.216	.222	.228	.234	.240	.245	.250	.256	.261	.266
.4	.271	.275	.280	.284	.289	.293	.297	.301	.305	.308
.5	.312	.315	.319	.322	.325	.328	.331	.334	.337	.339
.6	.342	.344	.346	.348	.350	.352	.354	.355	.357	.358
.7	.360	.361	.362	.363	.363	.364	.364	.365	.365	.365
.8	.365	.365	.364	.364	.363	.362	.361	.360	.359	.357
.9	.355	.353	.350	.348	.344	.341	.337	.332	.326	.318
1.0	.299									

Table 5.7

Determination of top width — Values of $\frac{T}{B}$

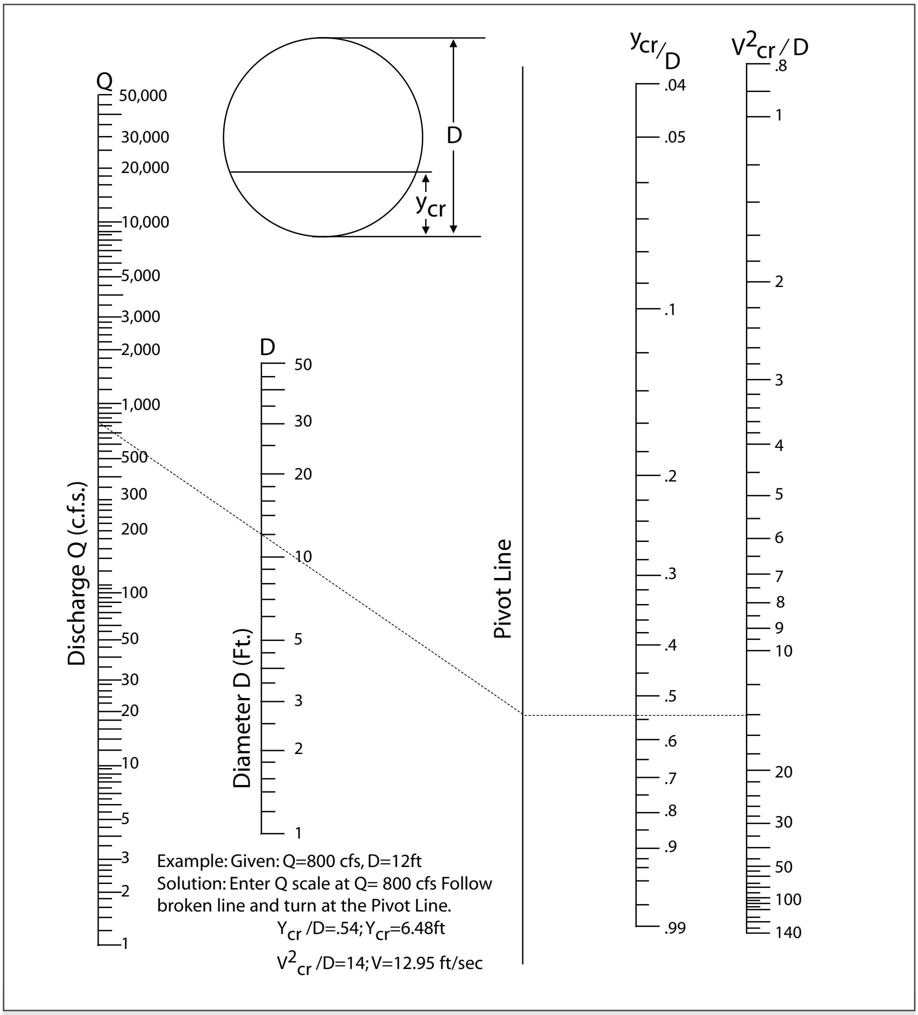
$\frac{y}{D}$	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
1		.900	.914	.927	.938	.948	.956	.964	.971	.976
.2	.982	.986	.990	.993	.995	.997	.998	.998	.998	.999
.3	.997	.996	.995	.993	.991	.989	.987	.985	.982	.979
.4	.976	.971	.967	.964	.960	.956	.951	.947	.942	.937
.5	.932	.927	.921	.916	.910	.904	.897	.891	.884	.877
.6	.870	.863	.855	.847	.839	.830	.822	.813	.803	.794
.7	.784	.773	.763	.752	.741	.729	.717	.704	.691	.678
.8	.664	.649	.634	.618	.602	.585	.567	.548	.528	.508
.9	.486	.462	.437	.410	.381	.349	.313	.272	.223	.158

The critical depth serves to distinguish two more classes of open channel flow:

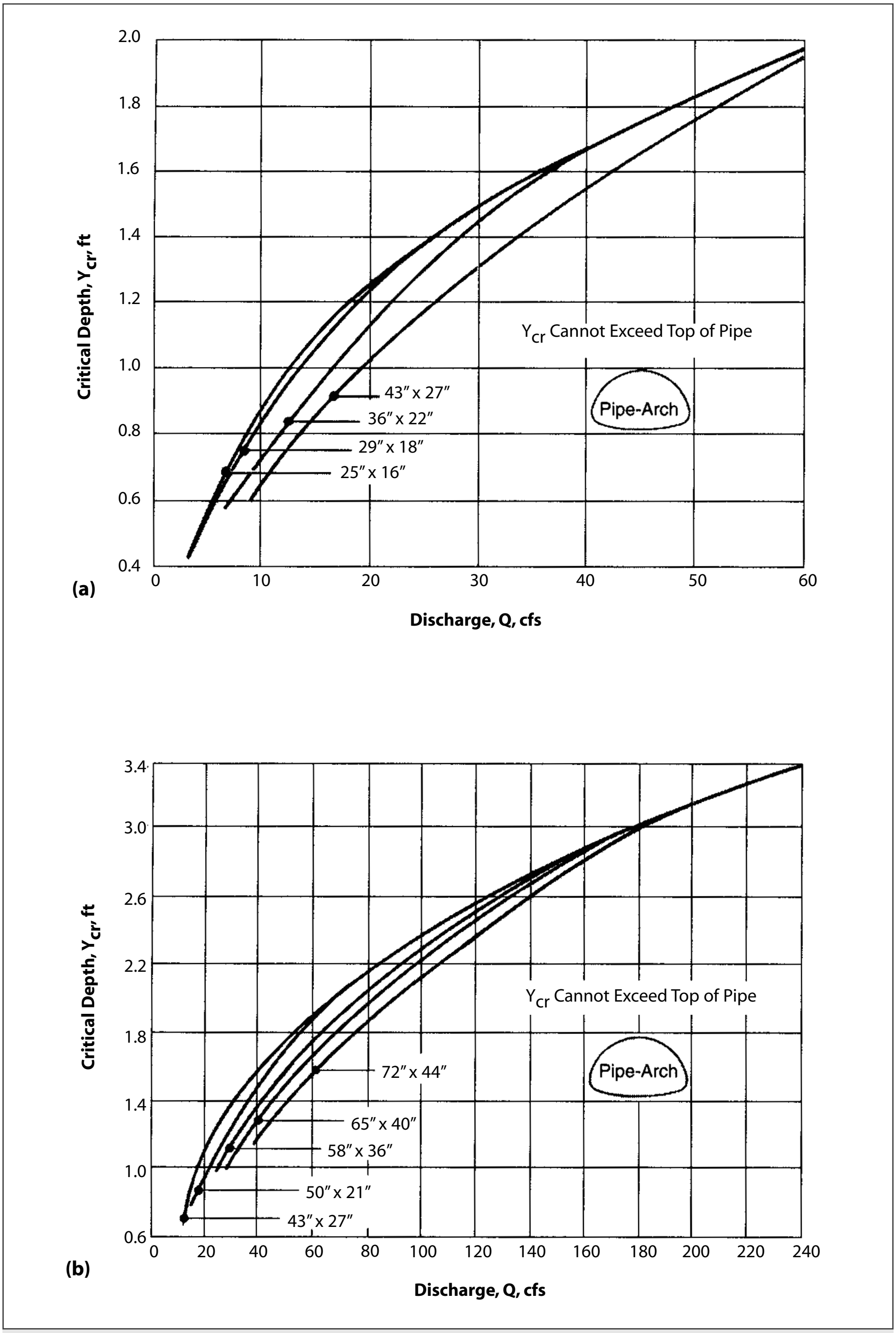
$y > y_{cr}$ - The specific energy is predominantly potential energy (y) and the kinetic energy is small. The velocity is less than V_{cr} and the flow is called SUBCRITICAL (i.e., with respect to velocity) or TRANQUIL.

$y < y_{cr}$ - Most of the specific energy is kinetic energy and the depth or potential energy is small. The velocity is greater than V_{cr} and the flow is therefore called SUPERCRITICAL or RAPID.

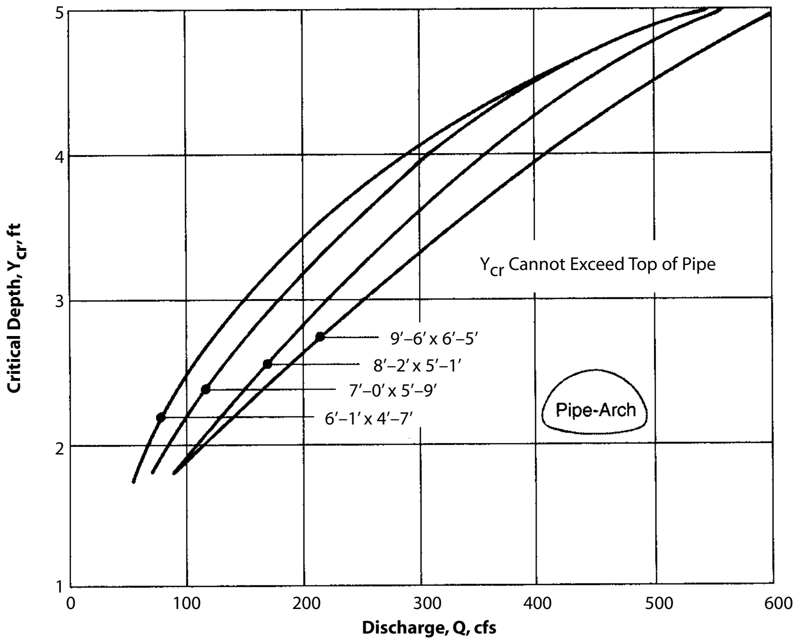
For circular pipes, Figure 5.5 provides a nomograph for calculating y_{cr}. For pipe arch CSP, Figures 5.6 and 5.7 provide a graphical method of determining critical flow depths.

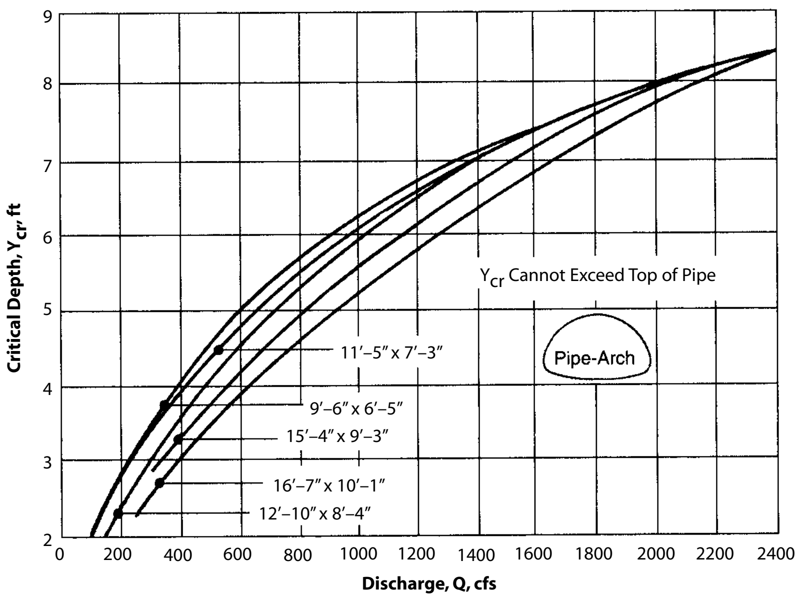


■ **Figure 5.5** Critical flow and critical velocity in circular pipes.



■ **Figure 5.6** Critical depth for standard corrugated steel pipe arches. (Adapted from Federal Highway Administration.)





■ **Figure 5.7** Critical depth for structural plate pipe arches. (Adapted from Federal Highway Administration.)

Energy Losses

When using the Bernoulli Equation for hydraulic design, it is necessary to make allowance for energy losses as illustrated in Figure 5.2. The losses are expressed in terms of head and may be classified as:

friction losses - losses due to the shear stress between the moving fluid and the boundary material; and

form losses - losses caused by abrupt transitions resulting from the geometry of manholes, bends, expansions and contractions.

It is a common mistake to include only friction losses in the hydraulic analysis. Form losses can constitute a major portion of the total head loss and, although estimates of form losses are generally based on empirical equations, it is important to make allowance for them in the design.

Friction Losses

In North America, the Manning and Kutter equations are commonly used to estimate the friction gradient for turbulent flow in storm sewers. In both equations, fully developed rough turbulent flow is assumed so that the head loss per unit length of pipe is approximately proportional to the square of the discharge (or velocity). Both equations use an empirical coefficient, typically termed Manning's 'n', to describe the roughness of the channel boundary. Table 5.8 provides suggested values of 'n' for a variety of pipes and channels. Tables 5.9 and 5.10 provide suggested values of 'n' for various corrugated steel pipe corrugation profiles and linings.

Table 5.8

Coefficient of roughness (Manning's n) for pipes and channels

Closed conduits	
Asbestos-cement pipe	0.011-0.015
Brick	0.013-0.017
Cast iron pipe	
Uncoated (new)	–
Asphalt dipped (new)	–
Cement-lined & seal coated	0.011-0.015
Concrete (monolithic)	
Smooth forms	0.012-0.014
Rough forms	0.015-0.017
Concrete pipe	0.011-0.015
Plastic pipe (smooth)	0.011-0.015
Vitrified clay	
Pipes	0.011-0.015
Liner plates	0.013-0.017
Open channels	
Lined channels	
a. Asphalt	0.013-0.017
b. Brick	0.012-0.018
c. Concrete	0.011-0.020
d. Rubble or riprap	0.020-0.035
e. Vegetal	0.030-0.400
Excavated or dredged	
Earth, straight and uniform	0.020-0.030
Earth, winding, fairly uniform	0.025-0.040
Rock	0.030-0.045
Unmaintained	0.050-0.140
Natural Channels (minor streams, top width at flood stage < 30m, 100 ft)	
Fairly regular section	0.030-0.0700
Irregular section with pools	0.040-0.100

Table 5.9

Coefficient of Roughness (Manning's n) for Standard Corrugated Steel Pipe

		2-2/3 x 1/2	Helical Corrugation, Pitch x Rise (in.)										
		Annular	1-1/2 x 1/4			2-2/3 x 1/2							
Flowing	Finish	Corrugation	Diameter (in.)										
		All Dia.	8	10	12	15	18	24	30	36	42	48	≥ 54
Full	Unpaved	0.024	0.012	0.014	0.011	0.012	0.013	0.015	0.017	0.018	0.019	0.020	0.021
Full	25% paved	0.021						0.014	0.016	0.017	0.018	0.020	0.019
Part Full	Unpaved	0.027			0.012	0.013	0.015	0.017	0.019	0.020	0.021	0.022	0.023
		All	Pipe Arch Span x Rise (in.)										
		Pipe Arches				17 x 13	21 x 15	28 x 20	35 x 24	42 x 29	49 x 33	57 x 38	≥ 54 x 43
Full	Unpaved	0.026				0.013	0.014	0.016	0.018	0.019	0.020	0.021	0.022
Part Full	Unpaved	0.029				0.018	0.016	0.021	0.023	0.024	0.025	0.025	0.026
		3 x 1	Helical Corrugation, Pitch x Rise (in.)										
		Annular	3 x 1										
		Corrugation	Diameter (in.)										
		All Dia.				36	42	48	54	60	66	72	≥78
Full	Unpaved	0.027				0.022	0.022	0.023	0.023	0.024	0.025	0.026	0.027
Full	25% Paved	0.023				0.019	0.019	0.020	0.020	0.021	0.022	0.022	0.023
		5 x 1	Helical Corrugation, Pitch x Rise (in.)										
		Annular	5 x 1										
		Corrugation	Diameter (in.)										
		All Dia.						48	54	60	66	72	≥78
Full	Unpaved	0.025						0.022	0.022	0.023	0.024	0.024	0.025
Full	25% Paved	0.022						0.019	0.019	0.020	0.021	0.021	0.022
			All Diameters										
Smooth Interior Pipe (1)			0.012										

Note (1): Includes fully paved, concrete lined, spiral rib pipe, ribbed pipe with inserts, and double wall pipe.

Table 5.10

Coefficient of roughness (Manning's n) for structural plate pipe (6 x 2 in. corrugations)

Corrugation	Diameters (ft)			
6 x 2 in.	5	7	10	15
Plain-unpaved	0.033	0.032	0.030	0.028
25% Paved	0.028	0.027	0.036	0.024

Manning Equation

The Manning Equation is one of a number of so-called empirical solutions. It is widely used for open channel flow calculations but can also be applied to closed pipe flow. The equation is as follows:

$$V = \frac{1.486}{n} R^{2/3} S_f^{1/2}$$

where: V = Average velocity, ft/s
R = Hydraulic radius = A/WP, ft
A = Cross-sectional area, ft^2
WP = Wetted perimeter, ft
S_f = Friction gradient or slope of energy line, ft/ft
n = Manning's roughness coefficient

The discharge, Q, is simply determined by multiplying the resulting velocity by the cross-sectional area of the pipe.

Figure 5.8 provides a nomograph for estimating steady uniform flow for pipe flowing full, using the Manning equation. In cases where pipes are flowing only partly full, the corresponding hydraulic section parameters may be determined from Figures 5.9 and 5.10. These figures provide hydraulic section parameters, as a fraction of the value for the pipe flowing full, for ratios of water depth to pipe diameter or rise.

Kutter Equation

The Kutter Equation is used for open channel calculations in certain areas of the United States. It is an empirically derived relation between the Chezy coefficient 'C' and the Manning roughness coefficient 'n'. The equation is as follows:

$$Q = A \bullet C \bullet R^{1/2} \bullet S_f^{1/2}$$

where
$$C = \frac{23 + \frac{0.00155}{S_f} + \frac{1}{n}}{1 + \frac{n}{\sqrt{R}}\left(23 + \frac{0.00155}{S_f}\right)}$$

Although the friction slope, S_f, appears as a second order term in the expression for 'C', the resulting discharge is not sensitive to this term. Table 5.11 shows the difference (%)

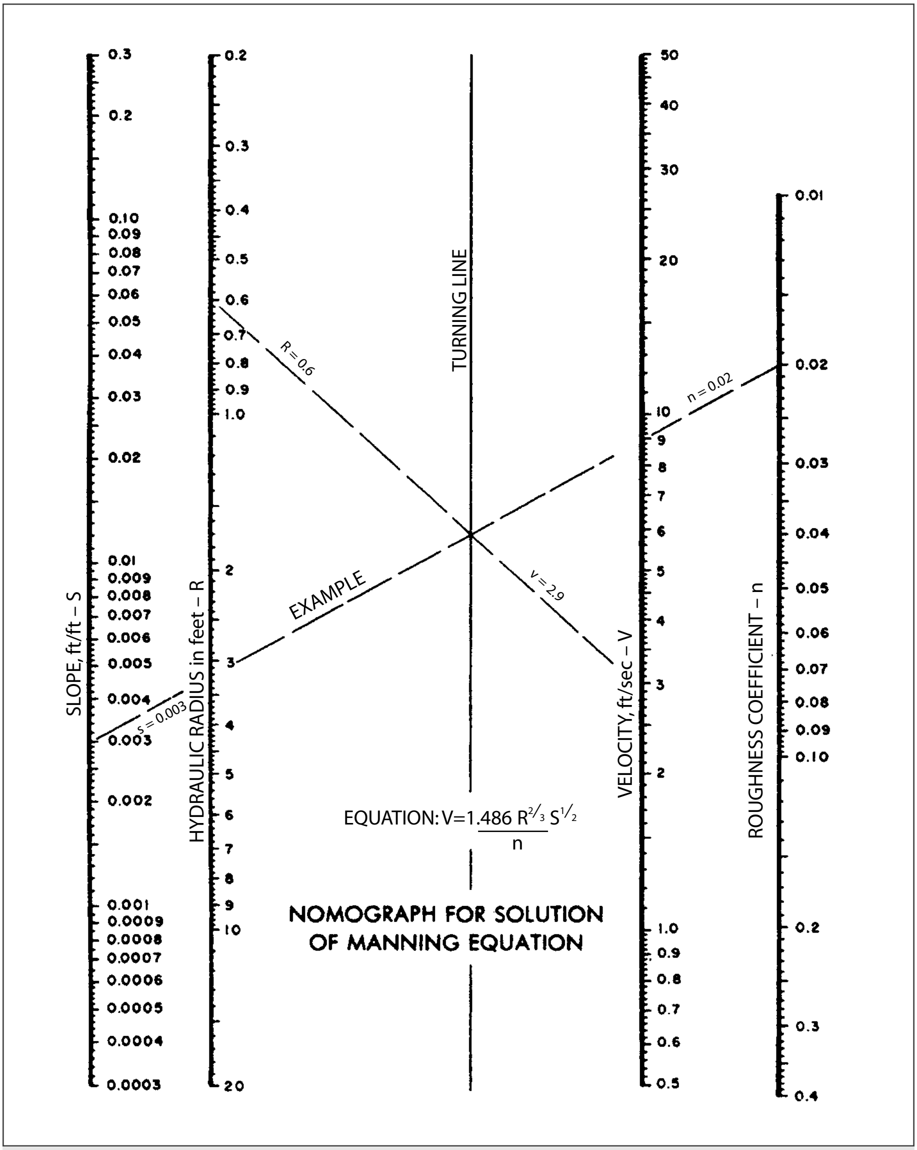


Figure 5.8 Nomograph for solution of Manning's formula.

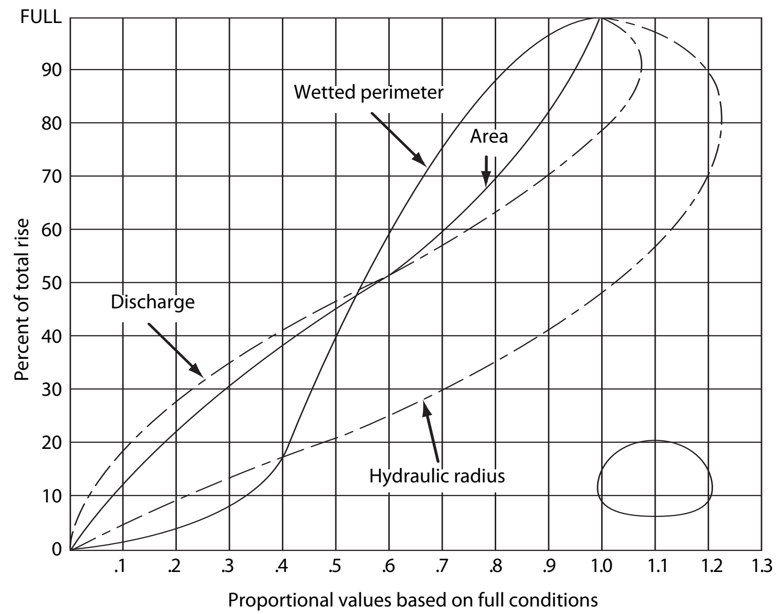


Figure 5.9 Hydraulic section parameters for corrugated steel and structural plate pipe arches.

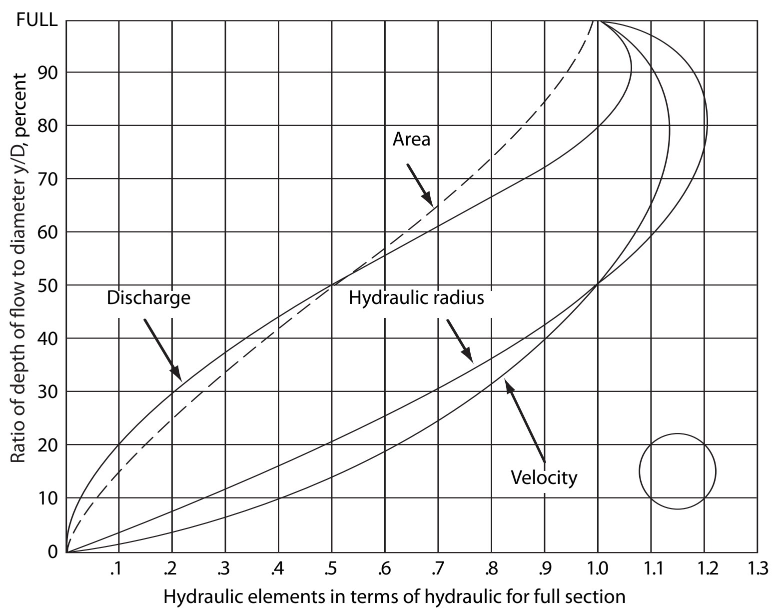


Figure 5.10 Hydraulic section parameters for circular corrugated steel and structural plate corrugated steel pipes.

in discharge computed using the Kutter equation compared with that obtained using the Manning equation. The table gives the relationship between the diameter, D, and the hydraulic radius, R, assuming full flow in a circular pipe. The values in Table 5.11 are also valid for noncircular pipes flowing partially full.

Table 5.11

Percent Difference of Kutter Equation Compared With Manning Equation (Grade = 1.0%)

Diameter D-(ft)	Hydraulic Radius R-(ft)	n = 0.013	n = 0.020	n = 0.030
1.0	0.25	-4.46	-16.18	-26.13
2.0	0.50	-0.46	-8.54	-16.74
3.0	0.75	2.05	-5.07	-11.82
4.0	1.00	2.58	-3.12	-8.70
5.0	1.25	2.66	-1.94	-6.54
6.0	1.50	2.51	-1.18	-4.95
7.0	1.75	2.25	-0.70	-3.74
8.0	2.00	1.92	-0.39	-2.80
9.0	2.25	1.55	-0.20	-2.05
10.0	2.50	1.17	-0.10	-1.45
11.0	2.75	0.78	-0.06	-0.96
12.0	3.00	0.38	-0.07	-0.56
13.0	3.25	-0.01	-0.12	-0.23
14.0	3.50	-0.39	-0.19	0.04
15.0	3.75	-0.77	-0.28	0.26
16.0	4.00	-1.14	-0.39	0.44

The two equations give identical results for values of R close to 3 feet, which represents a very large pipe of perhaps 144 inches diameter. For smaller sized pipes, the difference is significant, especially where the roughness coefficient is large.

Solving the Friction Loss Equation

Of the three parameters of greatest interest in open channel flow analysis (Q, S_f, y_o), the discharge, Q, and the friction slope, S_f, are easily obtained as they appear explicitly in the equations. Because of the exponential form of the Manning equation, it is a simple matter to compute the friction slope as a function of either velocity or discharge for known cross-sectional properties. Even with the Kutter equation, the second order term is of little importance and can be safely ignored as a first iteration when solving for S_f.

The third parameter of interest is the normal depth, y_o, which is the depth at which uniform flow would take place in a very long reach of channel. The normal depth is more difficult to determine as it appears in the expressions for both area, A, and hydraulic radius, R. A trial and error solution is required, except for sections of straightforward geometry.

For partially-full circular channels, a convenient semi-graphical method of solution is provided by the curves describing proportional ratios of discharge, hydraulic radius, area and velocity, expressed as a function of the relative depth y/D. The following two simple examples show how these curves can be used:

Example 1: Finding the normal depth.

A pipe with a diameter of 3 feet and a Manning's n of 0.013 has a gradient of 1.0%. Find the normal depth, y_o, for a discharge of 40 ft^3/s.

Step 1: Calculate the full-pipe capacity using Manning's equation for D = 3 ft
$R = D/4 = 0.75$ ft
$Q = 1.486 / n \bullet R^{2/3} \bullet S^{1/2} \bullet (\pi \bullet D^2 / 4) = 1.486 / 0.013 \bullet (0.75)^{2/3} \bullet (0.01)^{1/2} \bullet (\pi \bullet (3)^2 / 4) = 66.7\ ft^3/s$

Step 2: Calculate the proportional discharge
$Q_{act}/Q_{full} = 40 / 66.7 = 0.60$

Step 3: From the Discharge curve of Figure 5.10, the proportional depth, y/D, is 0.56 for a proportional discharge of 0.60.
The normal depth, y_o, is 0.56 * 3 = 1.68 ft

Example 2: Design for a range of flows.

A pipe, with a Manning's n of 0.013, is designed to carry a minimum discharge of 4.24 ft^3/s with a velocity not less than 3.28 ft/s, and a maximum discharge of 21.2 ft^3/s without surcharging. Use the flattest gradient possible. Find the pipe diameter and slope to satisfy these design criteria.

Step 1: Assume $Q_{full} = Q_{max} = 21.2$
Then $Q_{min} / Q_{full} = 4.24 / 21.2 = 0.2$

Step 2: From the Discharge curve of Figure 5.10, this corresponds to a proportional depth, y/D, of 0.30, which in turn corresponds to a proportional velocity (from the Velocity curve) of V_{min} / V_{full} of 0.78. The full pipe velocity, corresponding to the specified minimum velocity of 3.28 ft/s, is 3.28 / 0.78 = 4.21 ft/s

Step 3: For full pipe flow, the required section area is given by:
$A = Q_{max} / V_{full} = 21.2 / 4.21 = 5.04\ ft^2$
or $D = (4 \bullet A / \pi)^{1/2} = (4 \bullet 5.04 / \pi)^{1/2} = 2.53$ ft = 30.4 in.

Step 4: Assuming that the next smallest commercial size is 30 inches, the selected diameter must be rounded down to 30 inches to ensure that the minimum velocity is greater than 3.28 ft/s.

Step 5: The necessary slope is then obtained from the Manning equation as

$$S_o = S_f = \frac{Q^2n^2}{A^2R^{4/3}(1.486)^2}$$

where $A = \pi \bullet D^2 / 4 = \pi \bullet (2.5)^2 / 4 = 4.91 \text{ ft}^2$
and $R = D / 4 = 2.5 / 4 = 0.625$ ft
The required grade is $S_o = Q^2 \bullet n^2 / (1.486)^2 / A^2 / R^{4/3} =$
$(21.1)^2 \bullet (0.013)^2 / (1.486)^2 / (4.91)^2 / (0.625)^{4/3} = 0.0019$ or 1.9%

Water Surface Profiles

Uniform flow is seldom attained except in very long reaches, free from any form of transition. Gradually varied flow occurs as a form of gentle transition from one stage of uniform flow to another, and non-uniform flow is found to be the rule rather than the exception.

The flow profiles of gradually varied flow can be classified in relation to the normal depth, y_o, the critical depth, y_{cr}, and the slope of the channel.

Channel slope is described as:

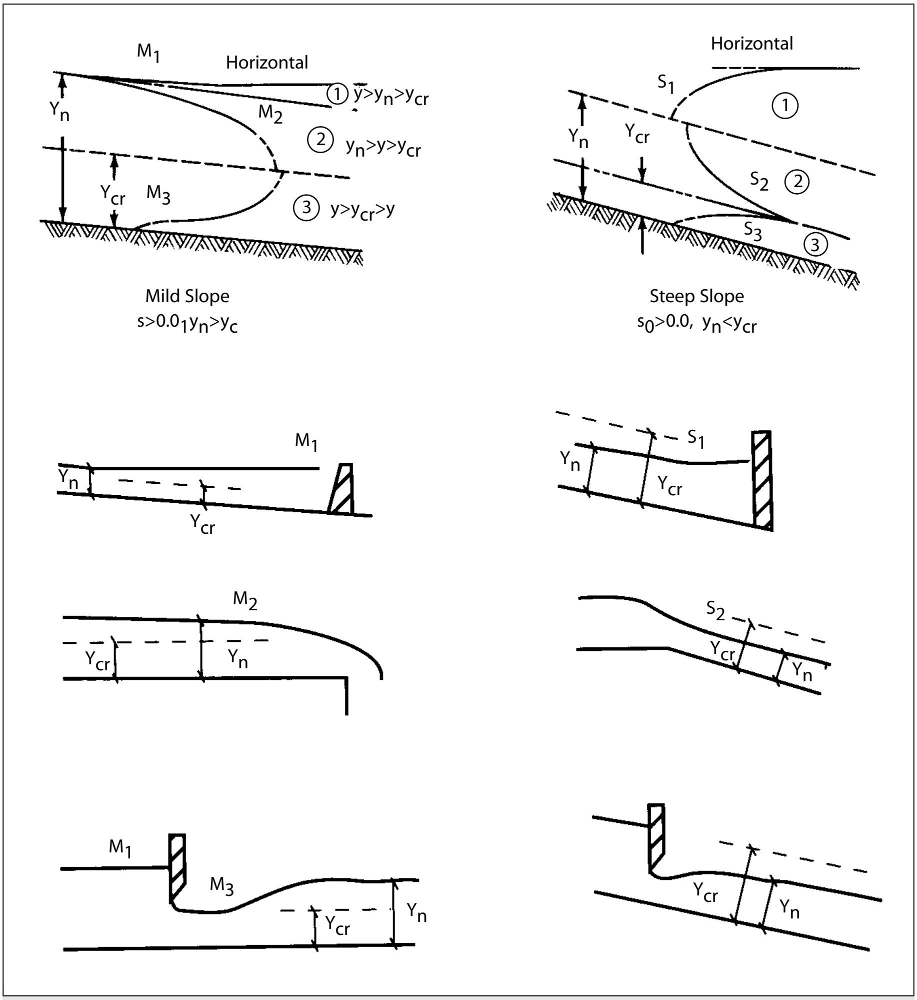
(1) MILD when $y_o > y_{cr}$ i.e. $S_o < S_{cr}$
(2) STEEP when $y_o < y_{cr}$ i.e. $S_o > S_{cr}$

Note that the critical slope, S_{cr}, is slightly dependent on the stage or magnitude of flow, so that strictly speaking the description of Mild or Steep should not be applied to the channel without regard to the flow conditions.

Most textbooks show five classes of channel slope: Mild, Steep, Critical, Horizontal and Adverse. In practice, the last three categories are special cases of the first two and it is sufficient to consider them. In addition to the channel slope, a profile of gradually varied flow can be classified depending on whether it lies above, below or between the normal and critical depths. The three zones may be defined as follows.

Zone 1 — Profile lies above both y_o and y_{cr}
Zone 2 — Profile lies between y_o and y_{cr}
Zone 3 — Profile lies below both y_o and y_{cr}

Using 'M' and 'S' to denote Mild or Steep channel and the Zone numbers '1', '2' or '3', profiles may be classified as 'M1' or 'S3', for example. Figure 5.11 shows the idealized cases of the six basic profile types along with typical circumstances in which they can occur.



■ **Figure 5.11** Idealized flow profiles.

Hydraulic Jump

When supercritical flow enters a reach in which the flow is subcritical, an abrupt transition is formed that takes the form of a surface roller or undular wave. The wave tries to move upstream but is held in check by the velocity of the supercritical flow. Figure 5.12 shows a typical situation in which supercritical uniform flow from a steep reach enters a reach of mild slope in which the normal depth is subcritical.

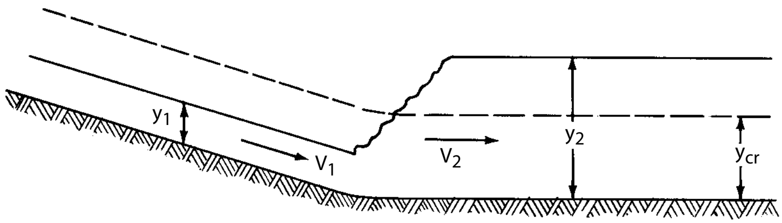


Figure 5.12 Hydraulic jump.

The energy losses associated with the violent turbulence of the hydraulic jump make application of the Bernoulli equation impossible. Instead, the control volume of fluid containing the jump can be analyzed using the equation of conservation of momentum. For a prismatic channel of arbitrary cross-section, this can be expressed as follows:

$$\frac{Q^2}{gA_1} + A_1 y_1 = \frac{Q^2}{gA_2} + A_2 y_2$$

where: y = Depth to the centroid of the cross-section, ft
A = Cross-sectional area, ft^2
Q = Total discharge, ft^3/s
g = Gravitational constant = 32.2 ft/s^2

For the special case of a rectangular cross-section, the solution can be obtained directly using the discharge per unit width:

$$y_2 = -\left(\frac{y_1}{2}\right) + \left(\frac{y_1^2}{4} + \frac{2q^2}{gy_1}\right)^{\frac{1}{2}}$$

where: y_2 = Depth downstream of the jump, ft
y_1 = Depth upstream of the jump, ft
q = Discharge per unit breadth of channel, $ft^3/s/ft$
g = Gravitational constant = 32.2 ft/s^2

The above equation is reversible so that y_1 may be found as a function of y_2 using a similar relationship.

Form Losses In Junctions, Bends And Other Structures

From the time storm water first enters the sewer system at the inlet until it discharges at the outlet, it will encounter a variety of hydraulic structures such as manholes, bends, contractions, enlargements and transitions, which will cause velocity head losses. These losses have sometimes been called "minor losses." This is misleading. In some situations these losses are as important as those arising from pipe friction. Velocity losses may be expressed in a general form derived from the Bernoulli and Darcy-Weisbach equations:

$$H = K\frac{V^2}{2g}$$

where:
- H = Velocity head loss, ft
- K = Structure specific coefficient
- V = Average velocity, ft/s
- g = Gravitational constant = 32.2 ft/s^2

The following are useful velocity head loss equations for hydraulic structures commonly found in sewer systems. They are primarily based on experiments.

Transition Losses (open channel)

The energy losses may be expressed in terms of the kinetic energy at the two ends:

$$H_t = K_t \, \Delta\left[\frac{V^2}{2g}\right]$$

where: K_t = Transition loss coefficient

Contraction:

$$H_t = .1\left(\frac{V_2^{\,2}}{2g} - \frac{V_1^{\,2}}{2g}\right) \qquad V_1 < V_2$$

where:
- V_1 = upstream velocity
- V_2 = downstream velocity

Expansion:

$$H_t = .2\left(\frac{V_1^{\,2}}{2g} - \frac{V_2^{\,2}}{2g}\right) \qquad V_1 > V_2$$

A simple transition from one pipe size to another, in a manhole with straight through flow, may be analyzed with the above equations.

Transition Losses (pressure flow)

Contraction:

$$H_t = K \left(\frac{V_2^2}{2g}\right) \left[1 - \left(\frac{A_2}{A_1}\right)\right]^2$$

where: K = 0.5 for sudden contraction
K = 0.1 for well designed transition
A_1, A_2 = cross-sectional area of flow of incoming and outgoing pipe from transition.

Expansion:

$$H_t = K \left[\frac{(V_1 - V_2)^2}{2g}\right]$$

where: K = 1.0 for sudden expansion
K = 0.2 for well designed transition

The above K values are for estimating purposes. If a more detailed analysis of the transition losses is required, then Tables 5.12 through 5.14, in conjunction with the energy losses equation in the form below, should be used for pressure flow.

$$H_t = K \left(\frac{V^2}{2g}\right)$$

Table 5.12

Values of K_2 for Determining Loss of Head Due to a Sudden Enlargement in a Pipe, for the Formula $H_2 = K_2 (V_1^2/2g)$

D_2/D_1 = Ratio of Larger Pipe to Smaller Pipe — V_1 = Velocity in Smaller Pipe

	Velocity, V_1, in feet per second												
D_2/D_1	2.0	3.0	4.0	5.0	6.0	7.0	8.0	10	12	15	20	30	40
1.2	.11	.10	.10	.10	.10	.10	.10	.09	.09	.09	.09	.09	.08
1.4	.26	.26	.25	.24	.24	.24	.24	.23	.23	.22	.22	.21	.20
1.6	.40	.39	.38	.37	.37	.36	.36	.35	.35	.34	.33	.32	.32
1.8	.51	.49	.48	.47	.47	.46	.46	.45	.44	.43	.42	.41	.40
2.0	.60	.58	.56	.55	.55	.54	.53	.52	.52	.51	.50	.48	.47
2.5	.74	.72	.70	.69	.68	.67	.66	.65	.64	.63	.62	.60	.58
3.0	.83	.80	.78	.77	.76	.75	.74	.73	.72	.70	.69	.67	.65
4.0	.92	.89	.87	.85	.84	.83	.82	.80	.79	.78	.76	.74	.72
5.0	.96	.93	.91	.89	.88	.87	.86	.84	.83	.82	.80	.77	.75
10.0	1.00	.99	.96	.95	.93	.92	.91	.89	.88	.86	.84	.82	.80
∞	1.00	1.00	.98	.96	.95	.94	.93	.91	.90	.88	.86	.83	.81

Table 5.13

Values of K_2 for Determining Loss of Head Due to a Gradual Enlargement in a Pipe, for the Formula $H_2 = K_2\ (V_1^2/2g)$

D_2/D_1 = Ratio of Diameter of Larger Pipe to Diameter of Smaller Pipe.
Angle of Cone is Twice the Angle Between the Axis of the Cone and its Side.

	Angle of Cone													
D_2/D_1	2°	4°	6°	8°	10°	15°	20°	25°	30°	35°	40°	45°	50°	60°
1.1	.01	.01	.01	.02	.03	.05	.10	.13	.16	.18	.19	.20	.21	.23
1.2	.02	.02	.02	.03	.04	.09	.16	.21	.25	.29	.31	.33	.35	.37
1.4	.02	.03	.03	.04	.06	.12	.23	.30	.36	.41	.44	.47	.50	.53
1.6	.03	.03	.04	.05	.07	.14	.26	.35	.42	.47	.51	.54	.57	.61
1.8	.03	.04	.04	.05	.07	.15	.28	.37	.44	.50	.54	.58	.61	.65
2.0	.03	.04	.04	.05	.07	.16	.29	.38	.46	.52	.56	.60	.63	.68
2.5	.03	.04	.04	.05	.08	.16	.30	.39	.48	.54	.58	.62	.65	.70
3.0	.03	.04	.04	.05	.08	.16	.31	.40	.48	.55	.59	.63	.66	.71
∞	.03	.04	.05	.06	.08	.16	.31	.40	.49	.56	.60	.64	.67	.72

Table 5.14

Values of K_3 for Determining Loss of Head Due to a Sudden Contraction in a Pipe, for the Formula $H_3 = K_3\ (V_2^2/2g)$

D_2/D_1 = Ratio of Larger Pipe to Smaller Pipe V_2 = Velocity in Smaller Pipe

	Velocity, V_2, ft/s												
D_2/D_1	2.0	3.0	4.0	5.0	6.0	7.0	8.0	10	12	15	20	30	40
1.1	.03	.04	.04	.04	.04	.04	.04	.04	.04	.04	.05	.05	.06
1.2	.07	.07	.07	.07	.07	.07	.07	.08	.08	.08	.09	.10	.11
1.4	.17	.17	.17	.17	.17	.17	.17	.18	.18	.18	.18	.19	.20
1.6	.26	.26	.26	.26	.26	.26	.26	.26	.26	.25	.25	.25	.24
1.8	.34	.34	.34	.34	.34	.34	.33	.33	.32	.32	.31	.29	.27
2.0	.38	.38	.37	.37	.37	.37	.36	.36	.35	.34	.33	.31	.29
2.2	.40	.40	.40	.39	.39	.39	.39	.38	.37	.37	.35	.33	.30
2.5	.42	.42	.42	.41	.41	.41	.40	.40	.39	.38	.37	.34	.31
3.0	.44	.44	.44	.43	.43	.43	.42	.42	.41	.40	.39	.36	.33
4.0	.47	.46	.46	.46	.45	.45	.45	.44	.43	.42	.41	.37	.34
5.0	.48	.48	.47	.47	.47	.46	.46	.45	.45	.44	.42	.38	.35
10.0	.49	.48	.48	.48	.48	.47	.47	.46	.46	.45	.43	.40	.36
∞	.49	.49	.48	.48	.48	.47	.47	.47	.46	.45	.44	.41	.38

Entrance Losses

$$H = K_e \frac{V^2}{2g}$$

where: K_e = Entrance loss coefficient (Table 5.15)

Table 5.15

Entrance Loss Coefficients For Corrugated Steel Pipes or Pipe Arches

Inlet End of Culvert	Coefficient K_e
Projecting from fill (no headwall)	0.9
Headwall, or headwall and wingwalls square-edged	0.5
Mitered (beveled) to conform to fill slope	0.7
*End-section conforming to fill slope	0.5
Headwall, rounded edge	0.2
Beveled Ring	0.25

Notes: *End sections available from manufacturers.

Manhole Losses

Manhole losses, in many cases, comprise a significant percentage of the overall losses within a sewer system. Consequently, if these losses are ignored, or underestimated, the sewer system may surcharge and cause basement flooding or sewer overflows. Losses at sewer junctions are dependent upon flow characteristics, junction geometry and relative sewer diameters. General problems with respect to flow through junctions have been discussed by Chow, who concluded that the losses could be best estimated by experimental analysis as opposed to analytical procedures.

Marsalek, in a study for three junction designs, found the following:

a) In pressurized flow, the most important flow variable is the relative lateral inflow for junctions with more than two pipes. The losses increase as the ratio of the lateral discharge to main line discharge increases.
b) Among the junction geometrical parameters, the important ones are: relative pipe sizes, junction benching and pipe alignment. Base shape and relative manhole sizes were less influential.
c) Full benching to the crown of the pipe significantly reduces losses as compared to benching to the mid-section of the pipe or no benching.
d) In junctions where two lateral inflows occur, the head losses increase as the difference in flows between the two lateral sewers increases. The head loss is minimized when the lateral flows were equal.

Various experimental studies have been performed to estimate manhole losses. These works should be referred to whenever possible. In cases where no applicable results are available, the following may be used as a guideline to estimate manhole losses.

In a straight through manhole, where there is no change in pipe size, losses can be estimated by:

$$H_m = 0.05 \frac{V^2}{2g}$$

Losses at terminal manholes may be estimated by:

$$H_{tm} = \frac{V^2}{2g}$$

Losses at junctions where one or more incoming laterals occur may be estimated by combining the laws of pressure plus momentum:

$$H_j = K_j \frac{V^2}{2g}$$

where: H_j = Junction losses

Using the laws of pressure plus momentum:

$$(H_j + D_1 - D_2) \frac{(A_1 + A_2)}{2} = \frac{Q_2^{\,2}}{A_2 g} - \frac{Q_1^{\,2}}{A_1 g} - \frac{Q_3^{\,2}}{A_3 g} \cos \theta$$

where the variable numbering is as shown in Figure 5.13.

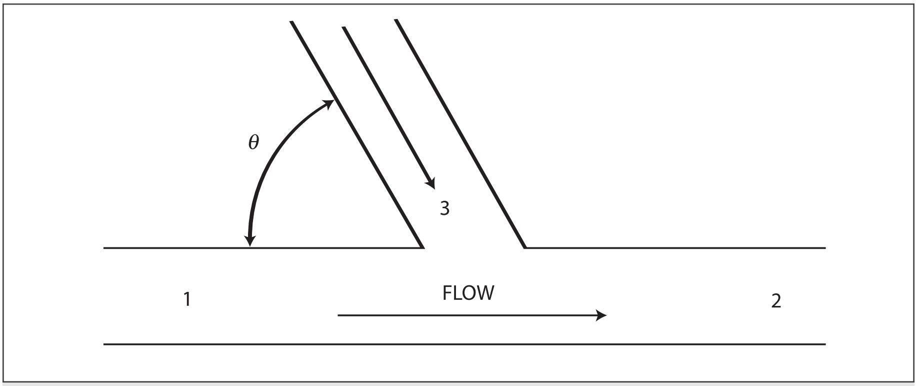


Figure 5.13 Manhole junction losses; variable numbering.

Bend Losses

Bend losses may be estimated from the equation:

$$H_b = K_b \frac{V^2}{2g}$$

For curved sewer segments where the angle is less than 40° the bend loss coefficient may be estimated as:

$$K_b = .25\sqrt{\frac{\Delta}{90}}$$

where: Δ = central angle of bend in degrees

For greater angles of deflection and bends in manholes, the bend loss coefficient may be determined from Figure 5.14.

HYDRAULICS OF STORM WATER INLETS

Storm water inlets are the means by which storm runoff enters the sewer system. Their design is often neglected or receives very little attention during the design of storm drainage systems. Inlets play an important role in road drainage and storm sewer design because of their effect on both the rate of water removal from the road surface and the degree of utilization of the sewer system.

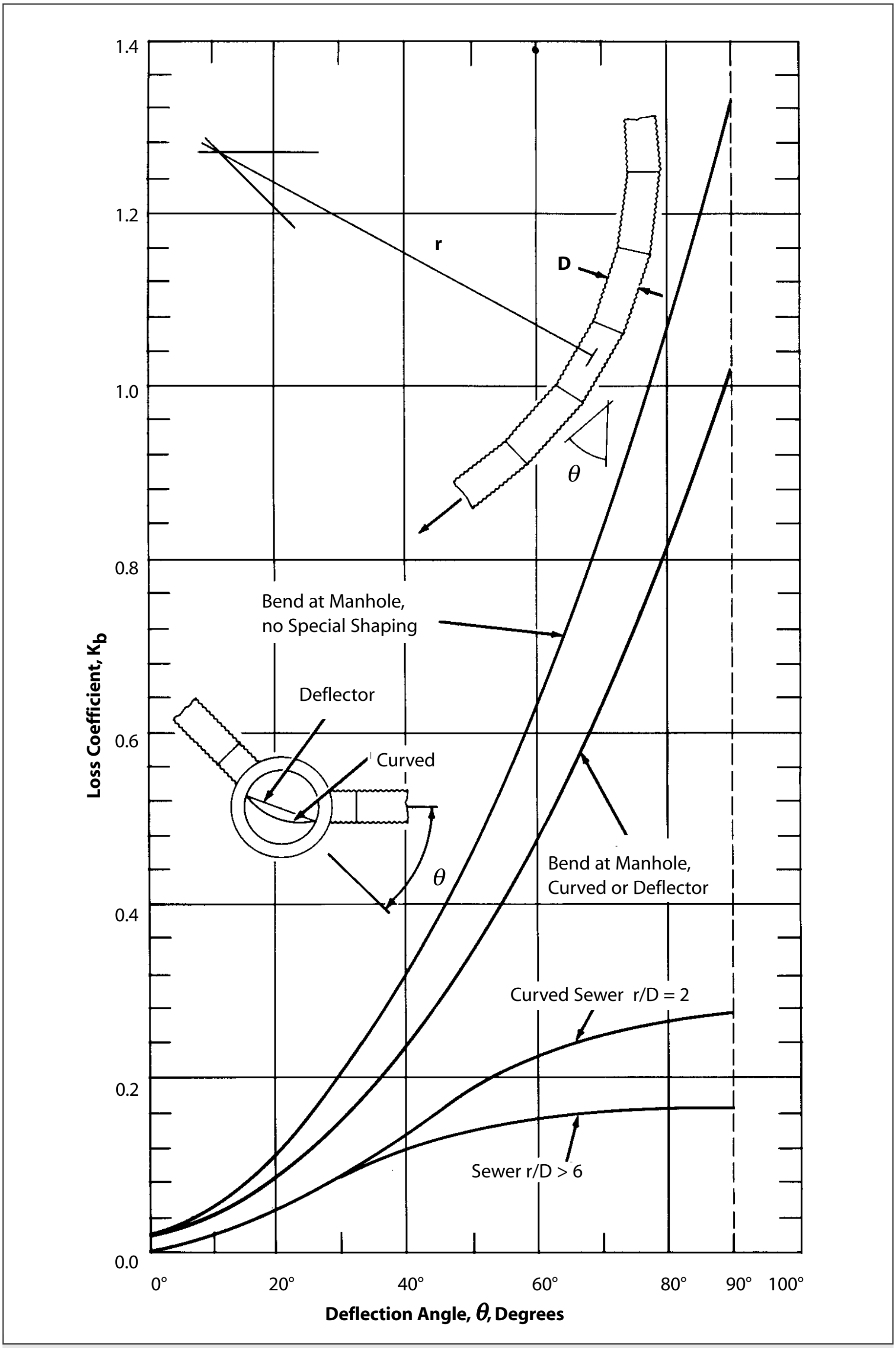
If inlets are unable to accept the design inflow into the sewer system, it may result in a lower level of roadway convenience and conditions hazardous to traffic. It may also lead to overdesign of the sewer pipes downstream of the inlet. In some cases, however, the limited capacity of the inlets may be desirable as a storm water management alternative, thereby offering a greater level of protection from excessive sewer surcharging. In such cases, both the quantity of runoff intercepted and the resulting level of roadway convenience must be known. Furthermore, overdesign in the number of inlets results in higher costs and could result in overuse of the sewer system.

It is imperative that more emphasis be placed on inlet design to assure that the inlet type, location and capacity are adequately determined to achieve the overall drainage requirements.

No one inlet type is best suited for all conditions. Many different types of inlets have thus been developed, as shown in Figure 5.15. In the past, the hydraulic capacities of some of these inlets were often unknown, sometimes resulting in erroneous capacity estimates.

Storm water inlets may not intercept all runoff due to the velocity of flow over the inlet and the spread of flow across the roadway and gutter. This leads to the concept of carryover flow. As carryover flow progresses downstream, it may accumulate, resulting in a greater demand for interception.

The hydraulic efficiency of inlets is a function of street grade, cross slope, inlet geometry, and curb and gutter design. Generally, an increased street cross-slope will result in increased inlet capacity as the flow is concentrated within the gutter. The depth of flow in the gutter may be estimated from Figure 5.16.



■ **Figure 5.14** Sewer bend loss coefficient.

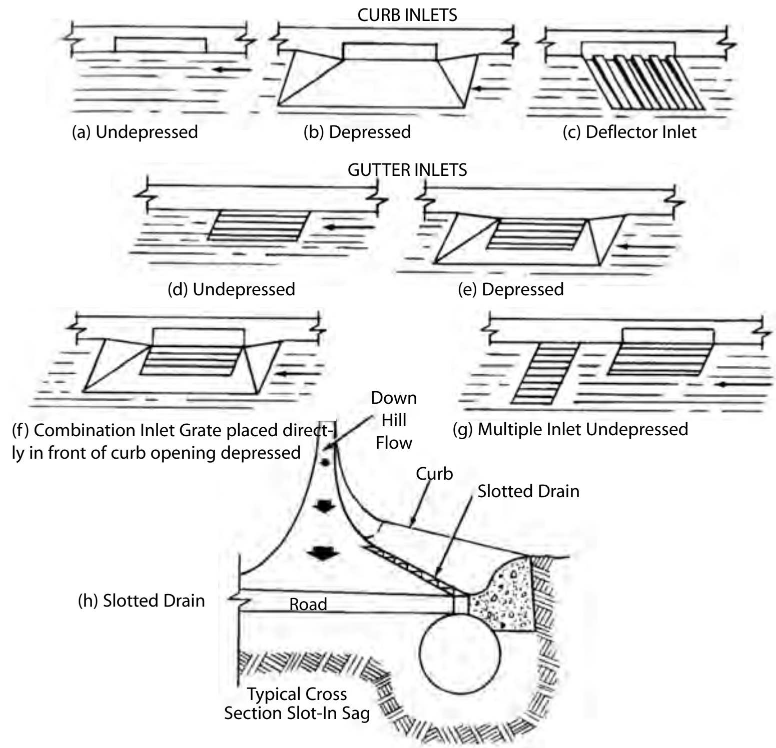
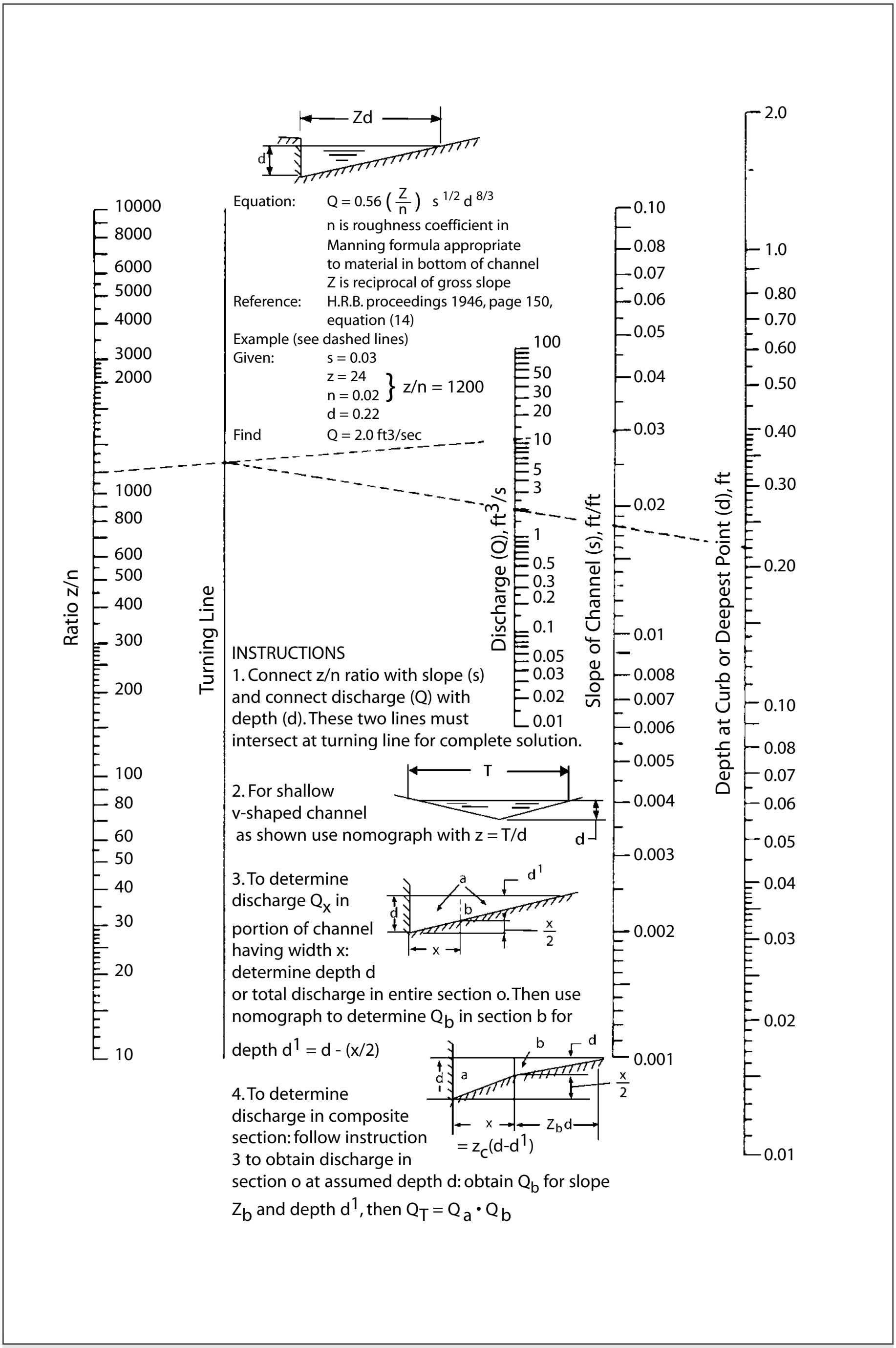


Figure 5.15 Storm water inlet types.

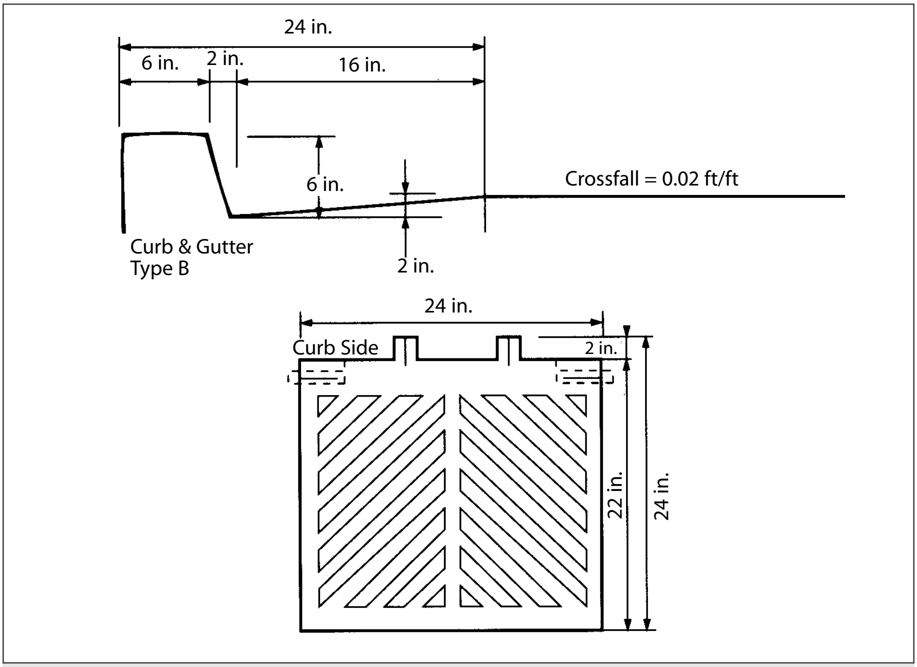
The effect of street grades on inlet capacities varies. Initially as the street grade increases there is an increase in gutter flow velocity, which allows a greater flow to reach the inlets for interception. However, as street grades continue to increase, there is a threshold where the velocity is so high that less flow can be intercepted. This threshold velocity depends upon the geometry of the inlet and characteristics of the gutter.

Experimental determination of inlet capacities have resulted in a set of tables and charts to aid the designer in storm water inlet selection and sewer system design. A sample of the results is shown in Figures 5.17 and 5.18 and Tables 5.16 and 5.17.

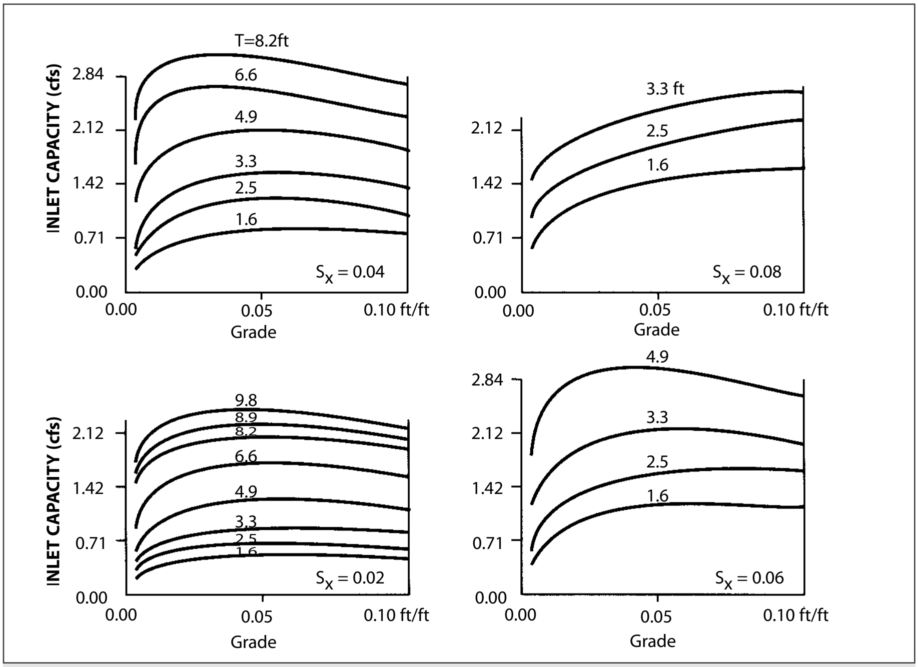
To use these charts or tables, the designer determines the overland flow and the resulting spread in gutter flow from a pre-determined road grade and crossfall, gutter design and inlet type (Table 5.16). This value is then used with Table 5.17 to obtain the storm water



■ **Figure 5.16** Nomograph for flow in triangular channels.



■ **Figure 5.17** Curb and gutter catch basin grate.



■ **Figure 5.18** Sewer inlet capacity for curb and gutter catch basin grate shown in Figure 5.17.

Table 5.16

Gutter Flow Rate (cfs)

Crossfall	Spread	Depth	Grade (ft/ft)							
(ft/ft)	(ft)	(ft)	0.003	0.01	0.02	0.03	0.04	0.06	0.08	0.10
	0.00	0.16	0.16	0.29	0.41	0.50	0.58	0.71	0.81	0.91
	1.64	0.20	.027	0.49	0.69	0.84	0.98	1.19	1.38	1.54
	2.46	0.21	0.35	0.64	0.90	1.11	1.28	1.56	1.81	2.02
	3.28	0.23	0.46	0.83	1.18	1.44	1.66	2.04	2.35	2.63
0.02	4.92	0.26	0.76	1.38	1.95	2.39	2.76	3.38	3.90	4.36
	6.56	0.30	1.19	2.18	3.08	3.78	4.36	5.34	6.17	6.89
	8.20	0.33	1.80	3.28	4.64	5.68	6.56	8.03	9.28	10.37
	8.86	0.34	2.09	3.81	5.39	6.60	7.62	9.34	10.78	12.05
	9.84	0.36	2.58	4.72	6.67	8.17	9.43	11.55	13.34	14.92
	1.64	0.23	0.41	0.76	1.07	1.31	1.51	1.86	2.14	2.39
	2.46	0.26	0.64	1.16	1.64	2.01	2.32	2.84	3.28	3.66
0.04	3.28	0.30	0.93	1.70	2.41	2.95	3.41	4.17	4.82	5.39
	4.92	0.36	1.81	3.31	4.69	5.73	6.63	8.11	9.37	10.47
	6.56	0.43	3.14	5.74	8.11	9.94	11.47	14.05	16.23	18.14
	8.20	0.49	5.00	9.12	12.90	15.80	18.24	22.34	25.80	28.84
	1.64	0.26	0.59	1.08	1.53	1.88	2.17	2.66	3.07	3.43
	2.46	0.31	1.00	1.82	2.58	3.15	3.64	4.46	5.15	5.76
0.06	3.28	0.36	1.56	2.84	4.02	4.93	5.69	6.96	8.04	8.99
	4.92	0.46	3.24	5.92	8.37	10.25	11.84	14.50	16.75	18.72
	5.48	0.49	3.99	7.29	10.31	12.63	14.59	17.86	20.63	23.06
	1.64	0.30	0.80	1.47	2.07	2.54	2.93	3.59	4.14	4.64
0.08	2.46	0.36	1.43	2.61	3.69	4.52	5.22	6.39	7.38	8.25
	3.28	0.43	2.31	4.23	5.98	7.32	8.45	10.35	11.95	13.36
	4.10	0.49	3.49	6.38	9.02	11.05	12.76	15.62	18.04	20.17

Table 5.17

Catch Basin Grate Inlet Capacity (cfs)

Crossfall	Spread	Depth	Grade (ft/ft)						
(ft/ft)	(ft)	0.00	0.01	0.02	0.03	0.04	0.06	0.08	0.10
	1.64	0.17	0.26	0.34	0.39	0.41	0.44	0.45	0.43
	2.46	0.28	0.41	0.50	0.59	0.63	0.66	0.68	0.61
	3.28	0.36	0.51	0.64	0.74	0.79	0.82	0.83	0.77
0.02	4.92	0.46	0.80	1.01	1.11	1.18	1.22	1.21	1.13
	6.56	0.81	1.25	1.42	1.53	1.55	1.54	1.51	1.45
	8.20	1.21	1.63	1.84	1.92	1.92	1.89	1.83	1.75
	8.86	1.29	1.77	1.97	2.03	2.04	2.02	1.96	1.84
	9.84	1.48	1.94	2.14	2.19	2.18	2.14	2.09	2.02
	1.64	0.24	0.45	0.60	0.69	0.76	0.84	0.83	0.75
	2.46	0.43	0.74	0.96	1.07	1.11	1.14	1.10	0.99
0.04	3.28	0.55	0.96	1.22	1.36	1.41	1.47	1.42	1.34
	4.92	0.97	1.63	1.90	2.01	2.04	1.98	1.87	1.77
	6.56	1.48	2.27	2.46	2.50	2.51	2.47	2.39	2.25
	8.20	2.03	2.75	2.85	2.85	2.82	2.70	2.59	2.54
	1.64	0.34	0.54	0.74	0.86	0.93	0.99	1.07	1.06
0.06	2.46	0.66	0.99	1.16	1.27	1.39	1.50	1.57	1.53
	3.28	1.07	1.49	1.69	1.83	1.90	1.96	1.94	1.80
	4.92	1.69	2.19	2.43	2.52	2.56	2.52	2.40	2.21
	1.64	0.46	0.81	1.04	1.14	1.24	1.33	1.35	1.34
0.08	2.46	0.96	1.33	1.49	1.61	1.73	1.89	2.00	2.02
	3.28	1.34	1.78	1.65	2.15	2.24	2.41	2.55	2.63

inlet or grate inlet capacity. The difference between the flow on the roadway and the inlet capacity is referred to as the carryover. An illustrative example is presented below:

Design Parameter Road crossfall = 0.02 ft/ft
Road grade = 0.02 ft/ft
Gutter type B
Inlet grate type per Figure 5.17
One inlet on each side of the road
There is no upstream carryover flow (0 ft^3/s)
Catchment Runoff = 6.2 ft^3/s

The flow in each gutter will be half of the total runoff.
Gutter Flow = 6.2 / 2 + 0 = 3.1 ft^3/s

From Table 5.16 the resulting spread in flow for a depth of 0.30 feet is 6.56 feet.

From Table 5.17, 6.56 feet of spread results in an inlet capacity of 1.42 ft^3/s.

Therefore, the total flow intercepted = 2 x 1.42 = 2.84 ft^3/s.

The carryover flow = 6.2 – 2.84 = 3.36 ft^3/s.

For roads where few restrictions to inlet location may exist (i.e., highways and arterial roads), these charts can be used to establish minimum spacing between inlets. This is done by controlling the catchment area for each inlet. The area is simplified to a rectangular shape of width and length where the length represents the distance between inlets.

Under special circumstances, it may be necessary to install twin or double inlets to increase the inlet capacity. For reasons of interference by traffic, such installations are usually installed in series, parallel to the curb. Studies have shown that where such installations exist on a continuous grade, the increases in inlet capacity rarely exceed 50 percent of the single inlet capacity.

The capacity of storm water inlets at a sag in the roadway is typically expressed by weir and orifice equations. Flow into the inlets initially operates as a weir having a crest length equal to the length of the inlet perimeter that the flow crosses. The inlet operates under these conditions to a depth of about 4 inches. The quantity intercepted is expressed by the following:

$$Q = C\,L\,D^{1.5}$$

where: Q = Rate of discharge into the grate opening, ft^3/s
C = 3.0
L = Perimeter length of the grate, disregarding bars and neglecting the side against the curb, ft
D = Depth of water at the grate, ft

When the depth exceeds 0.4 feet, the inlet begins to operate as an orifice and its discharge is expressed by the following:

$$Q = C\,A\,D^{0.5}$$

where: Q = Rate of discharge into the grate opening, ft^3/s
A = Clear opening of the grate, ft^2
C = 3.0
D = Depth of water ponding above the top of the grate, ft

The inlet capacity of an non-depressed curb inlet may be expressed by the equation:

$$Q/l = C\,(1x10^{-3})\,d\,(g/d)^{1/2}$$

where: Q = Discharge into inlet, ft^3/s
C = 4.82
l = Length of opening, ft
g = Gravitational constant = 32.2 ft/s^2
d = Depth of flow in gutter, ft

Another equation can be used, for the same calculation, which assumes a gutter of wedge shaped cross-section with a cross-sectional street slope of 10^{-3} to 10^{-1}:

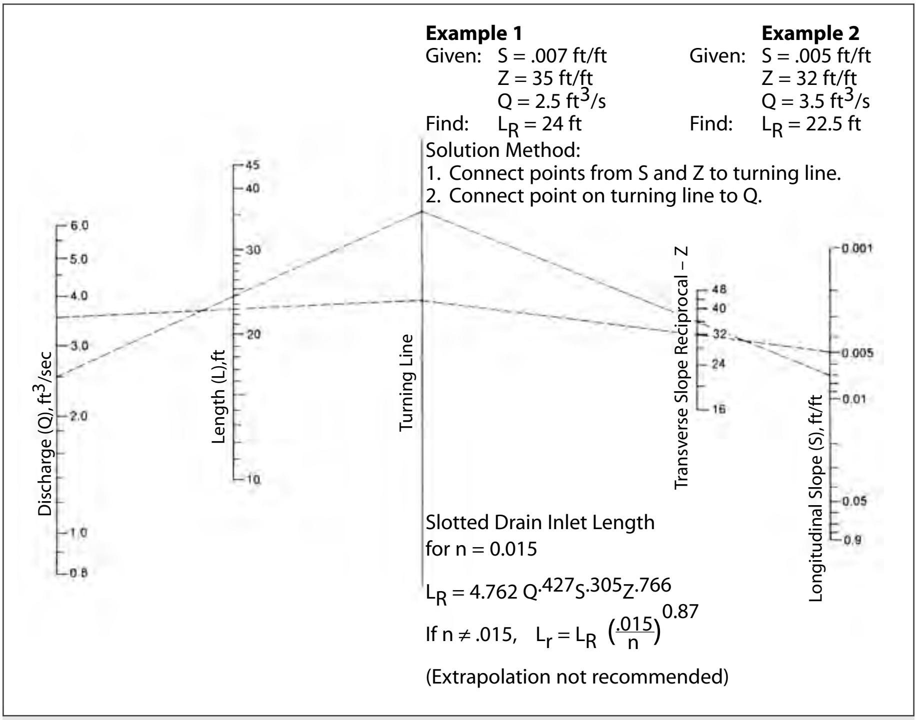
$$Q/l = C\,i^{0.579}\left(\frac{Q_o}{\sqrt{s/n}}\right)^{0.563}$$

where: Q_o = Flow in the gutter, ft^3/s
i = Transverse slope, ft/ft
s = Hydraulic gradient of gutter, ft/ft
n = Coefficient of roughness of gutter
C = 1.87

Slotted Drain

Slotted drain inlets are typically located as spaced curb inlets on a grade (sloping roadway) to collect downhill flow, or located in a sag (low point). The inlet capacity of a slotted drain may be determined from Figure 5.19,

where: S = Longitudinal gutter or channel slope, ft/ft
S_x = Transverse slope, ft/ft
Z = Transverse slope reciprocal = 1 / S_x, ft/ft
Q = Discharge, ft^3/s
L = Length of Slot, ft



■ **Figure 5.19** Slotted drain design nomograph.

It is suggested that the length of a slotted drain be in increments of 5 or 10 feet to facilitate fabrication, construction and inspection. Pipe diameter is usually not a factor but it is recommended that it be at least 18 inches

For a series of slotted drain curb inlets on a grade, each inlet will collect all or a major portion of the flow to it. The anticipated flow at points along the curb can be determined by the methods described previously in this chapter.

Once the initial upstream inlet flow is established, Figure 5.19 is used to determine the required length of slot to accommodate the total flow at the inlet.

The length of slot actually used may be less than required by Figure 5.19. Carryover is that portion of the flow that does not form part of the flow captured by the slotted drain. While some of the flow enters the drain, some flows past the drain to the next inlet. The efficiency of a slotted drain, required in order to consider carryover, is shown in Figure 5.20,

where: Q_d = Total discharge at an inlet, ft^3/s
Q_a = An assumed discharge, ft^3/s

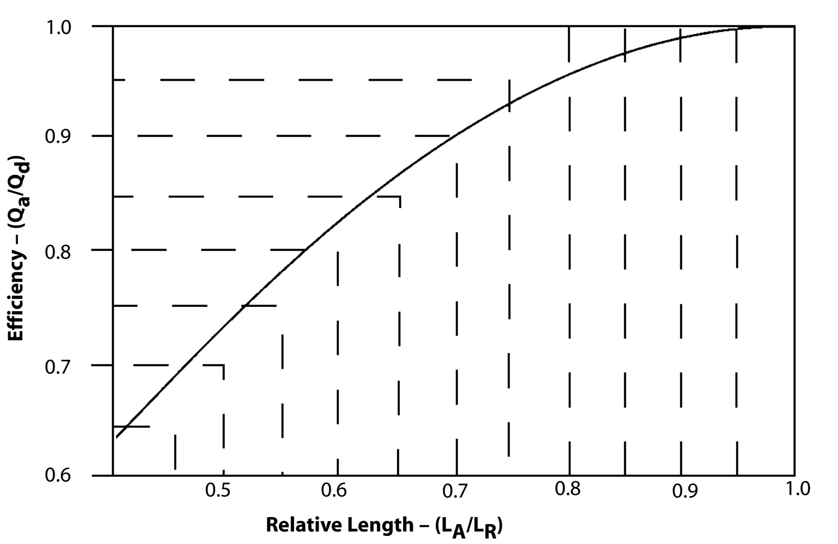


Figure 5.20 Slotted drain carryover efficiency.

If carryover is permitted, the designer must assume a length of slot such that the ratio of the assumed length of slot to the length of slot required for total interception and no carryover (L_A /L_R) is greater than 0.4 but less than 1.0. In other words, the designer must decide on a length of slot that will provide an acceptable carryover efficiency. Where carryover is not permitted, L_A must be at least the length L_R.

Economics usually favor slotted drain pipe inlets designed with carryover rather than for total flow interception. There must be a feasible location to which the carryover may be directed.

The actual length of slotted drain required, when carryover is allowed, can be determined using Figure 5.20. For example, if 20% carryover (slotted drain efficiency, Q_a / Q_d = 80%) is allowed, then only 58% (L_A/L_R) of the total slotted drain length is required, resulting in a 42% savings in material and installation costs.

The slotted drain efficiency can also be calculated using the following equation:

$$E = 1 - 0.918\left(1 - \frac{L_A}{L_R}\right)^{1.769}$$

where: E = Efficiency, fraction
L_A = Actual slot length, ft
L_R = Slot length required for no carryover, ft

The amount of carryover can be calculated using the following equation:

$$CO = Q_d\,(1 - E)$$

where: CO = Carryover flow, ft^3/s
Q_d = Total design flow, ft^3/s

Combining the above two equations results in the following equation for carryover flow:

$$CO = 0.918\,Q_d \left(1 - \frac{L_A}{L_R}\right)^{1.769}$$

At sag inlets, the required length of slotted drain for total interception should be based on the orifice equation, which is:

$$Q_d = C\,A\sqrt{2gd}$$

where: C = Orifice coefficient = 0.61
A = Open area of slot based on the width of the slot and the length L_R, ft^2
g = Gravitational constant = 32.2 ft/s^2
d = Maximum allowable depth of water in the gutter, ft

Solving for the required slot length, L_R, assuming a slot width of 1.75 inches, results in the following equation:

$$L_R = \frac{1.401\,Q_d}{\sqrt{d}}$$

For a slotted drain in a sag at the end of a series of drains on a grade, the flow to the drain will include any carryover from the immediately adjacent drain upgrade. Unlike a drain-on-grade situation, a slotted drain in a sag will produce significant ponding if its capacity will not accommodate the design flow. Therefore, the actual length of sag inlets should be at least 2 times the calculated required length. This helps ensure against a hazard caused by plugging from debris. L_A should never be less than 20 feet for sag inlet cases.

Carryover is not usually permitted at level grade inlets. In that case, the actual slotted drain length must be at least the required length.

In addition to applications where slotted drain is located parallel to a curb and gutter and in the gutter, it is also used effectively to intercept runoff from wide, flat areas such as parking lots, highway medians and even tennis courts and airport loading ramps. The water is not, in these applications, collected and channeled against a berm (or curb), but rather the drain is placed transverse to the direction of flow so that the open slot acts as a weir intercepting all of the flow uniformly along the entire length of the drain.

Slotted drain has been tested for overland flow (sheet flow). The tests included flows up to 0.04 ft^3/s per foot of slot. The slotted drain used in the test was designed to accommodate at least 0.025 ft^3/s per foot, which corresponds to a rainstorm of 15 in./hr over a 72 foot wide roadway (6 lanes). Slopes ranged from a longitudinal slope of 9% and a transverse slope reciprocal of 16 ft/ft, to a longitudinal slope of 0.5% and a transverse slope reciprocal of 48 ft/ft. At the design discharge of 0.025 ft^3/s per foot, it was reported that the total flow fell through the slot as weir flow, without hitting the curb side of the slot. Even at the maximum discharge of 0.04 ft^3/s per foot and maximum slopes, nearly all the flow passed through the slot.

HYDRAULICS OF SUBDRAINS

Ground water may be in the form of an underground reservoir or it may be flowing through a seam of pervious material. If it is flowing, it may be seeping or percolating through a seam between impervious strata, or be concentrated in the form of a spring.

Free water moves through the ground by gravity. It may consist of storm water seeping through cracks in the pavement or entering the ground along the edges of the pavement or road. It may be ground water percolating from a higher water-bearing stratum to a lower one, or from a water-bearing layer into the open as in the case of an excavation.

A number of subdrainage applications are discussed in Chapter 1.

Water seeping through cracks in the pavement is especially noticeable in springtime and also visible shortly after rains when the remainder of the road has dried. Passing traffic pumps some of this water, sometimes mixed with subgrade soil, up through the cracks or joints onto the road surface. This water is harmful because it may freeze on the surface and become an unexpected traffic hazard, and it can also destabilize the road subgrade. It can and should be removed in order to establish a stable subgrade and to prevent potential problems.

Subsurface Runoff Computation

In general, the amount of available ground water is equivalent to the amount of water that soaks into the ground from the surface less the amounts that are used by plants or lost by evaporation. The nature of the terrain and the catchment area size, shape and slopes, as well as the character and slopes of the substrata, are contributing factors to the amount of ground water available and the volume of subsurface runoff.

A practical way to determine the presence of ground water, and the potential flow rate, is to dig a trench or test pit. This is helpful especially where an intercepting drain is to be placed across a seepage zone to intercept the ground water and divert the flow, as shown in Figure 5.21.

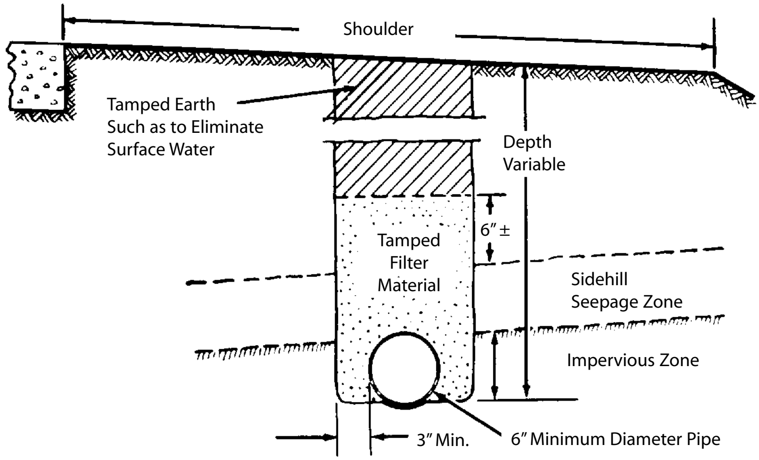


Figure 5.21 Intercepting Drain

Geotextiles are sometimes used in addition to, or instead of, free draining granular back-fill. The fabric serves as a filter, preventing fine erodible soils from entering the subdrain system while allowing the free flow of water. A typical cross-section of a trench design utilizing a geosynthetic filter fabric as a separator/filter is shown in Figure 5.22.

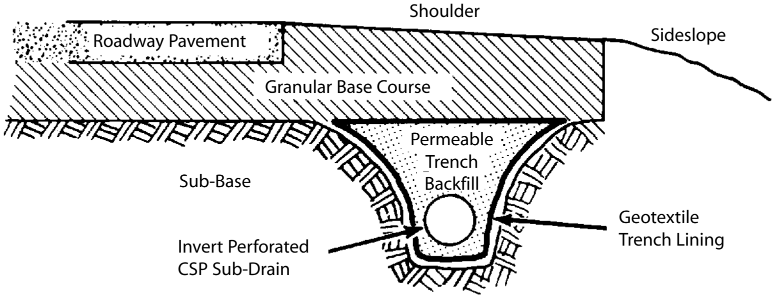


Figure 5.22 Intercepting Drain

Determining a correct size for subdrainage pipe requires an indirect approach. For problems other than those involving large flat areas, size determination becomes a matter of personal judgment and local experience. The following procedure applies to relatively flat areas.

The rate of runoff for average agricultural soils has been determined by agricultural engineering experiment stations to be about 0.375 inches in 24 hours. For areas of heavy rainfall or more pervious soils, this factor may be increased to 0.75 or 1.0 inch. The runoff expressed in inches per 24 hours is converted to $ft^3/s/acre$ for design discharge calculations. Table 5.18 provides a conversion table.

Table 5.18

Subsurface Runoff Constants for Various Soil Permeability Types

Soil Permeability Type	Depths of Water Removed in 24 Hours in.		Quantity of Water per Lateral, $ft^3/s/acre$ Constant C
	Fraction	Decimal	
Slow to Moderate	1/16	0.0625	0.0026
Slow to Moderate	1/8	0.1250	0.0052
Slow to Moderate	3/16	0.1875	0.0079
Slow to Moderate	1/4	0.2500	0.0105
Moderate	5/16	0.3125	0.0131
Moderate	3/8	0.3750	0.0157
Moderate	7/16	0.4375	0.0184
Moderate	1/2	0.5000	0.0210
Moderate	9/16	0.5625	0.0236
Moderate to Fast	5/8	0.6250	0.0262
Moderate to Fast	11/16	0.6875	0.0289
Moderate to Fast	3/4	0.7500	0.0315
Moderate to Fast	13/16	0.8125	0.0341
Moderate to Fast	7/8	0.8750	0.0367
Moderate to Fast	15/16	0.9375	0.0394
Moderate to Fast	1	1.0000	0.0420

The design discharge can be calculated from the following:

$$Q = CA$$

where: Q = Discharge or required capacity, ft^3/s
C = Subsurface runoff factor, $ft^3/s/acre$
A = Area to be drained, acres

Example:
Assuming a drainage runoff rate of 0.375 inches in 24 hours (runoff factor, C = 0.0157) and laterals 600 feet long spaced on 50 foot centers, the following result is obtained:

$$Q = 0.0157 \text{ x } \frac{600 \text{ x } 50}{43{,}560 \text{ ft}^2\text{/acre}} = 0.0108 \text{ ft}^3\text{/sec}$$

Size of Pipe

The size of pipe can be determined using Manning's formula, or by the use of a nomograph. For standard subdrainage applications, approximately 500 feet of 6 inch diameter perforated steel pipe may be used before increasing the pipe size to the next diameter.

Where possible, a minimum slope of 0.15 percent should be used for subdrainage lines. It is often permissible to use an even flatter slope to achieve a free outlet, but the steeper slope provides a self-cleansing flow velocity.

HYDRAULIC DESIGN OF STORM SEWERS

The hydraulic design of a sewer system may have to take into account the effect of backwater (the limiting effect on flows that a downstream sewer has on upstream sewers), surcharging, inlet capacity and all energy losses in the system. Whether each, or all, of these factors have to be considered depends on the complexity of the sewer system and the objectives of the analysis (i.e., whether the sizing of the system is preliminary or final). Furthermore, the degree of analysis will also depend on the potential impact should the sewer system capacity be exceeded. For example, consideration should be given to whether surcharging would result in damage to private property as a result of foundation drains being connected to the system, and whether the depth of flooding on a roadway would impact access by emergency vehicles that depend on safe access along the street. By evaluating the above factors, the designer will be in a position to select the level of analysis required.

The two hand calculation methods that follow assume all flows enter the sewer system. In other words, the inlet capacity of the system is not a limiting factor.

Flow charts and nomographs, such as those previously presented in this chapter, provide quick answers for the friction head losses in a given run of straight pipe between structures (manholes, junctions). These design aids do not consider the additional head losses associated with other structures and appurtenances common in sewer systems.

In most instances, when designing with common friction flow relationships such as the Manning equation, the hydraulic grade is assumed to be equal to the pipe slope at an elevation equal to the crown of the pipe. Consideration must therefore also be given to the changes in hydraulic grade line due to pressure changes, elevation changes, manholes and junctions. The design should not only be based on the pipe slope, but on the hydraulic grade line.

A comprehensive storm sewer design must therefore proceed on the basis of one run of pipe or channel at a time, progressing with the design methodically through the system. Only in this way can the free flow conditions be known and the hydraulic grade controlled, thus assuring performance of the system.

Making such an analysis requires backwater calculations for each run of pipe. This is a detailed process, which is demonstrated on the following pages. However, it is recognized that a reasonable conservative "estimate" or "shortcut" will sometimes be required. This can be done and is demonstrated in a discussion titled, "Method of Determining Equivalent Hydraulic Alternatives".

When using the backwater curve approach, the designer should first establish the type of flow (sub-critical or supercritical) to determine the direction in which the calculations should proceed.

- Super critical flow – a designer works downstream with flow
- Sub-critical flow – a designer works against the flow
- Note that a hydraulic jump may form if there is super and sub-critical flow in the same sewer

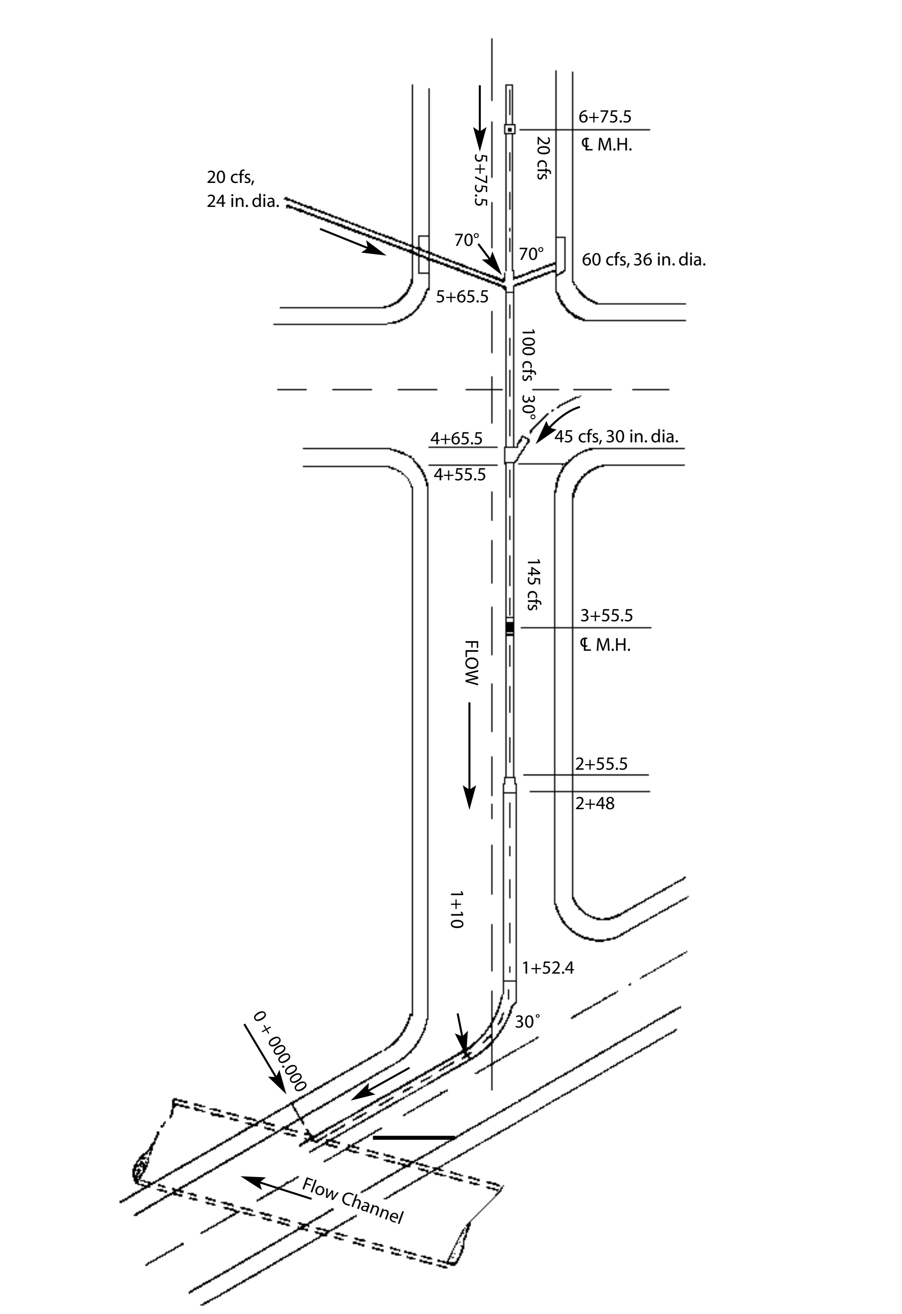
Backwater Analysis

A plan and profile of a storm drainage system is presented in Figures 5.23 and 5.24. A flow profile is shown where the hydraulic grade is set at the crown of the outlet pipe. Hydrological computations have been made, and preliminary design for the initial pipe sizing has been completed.

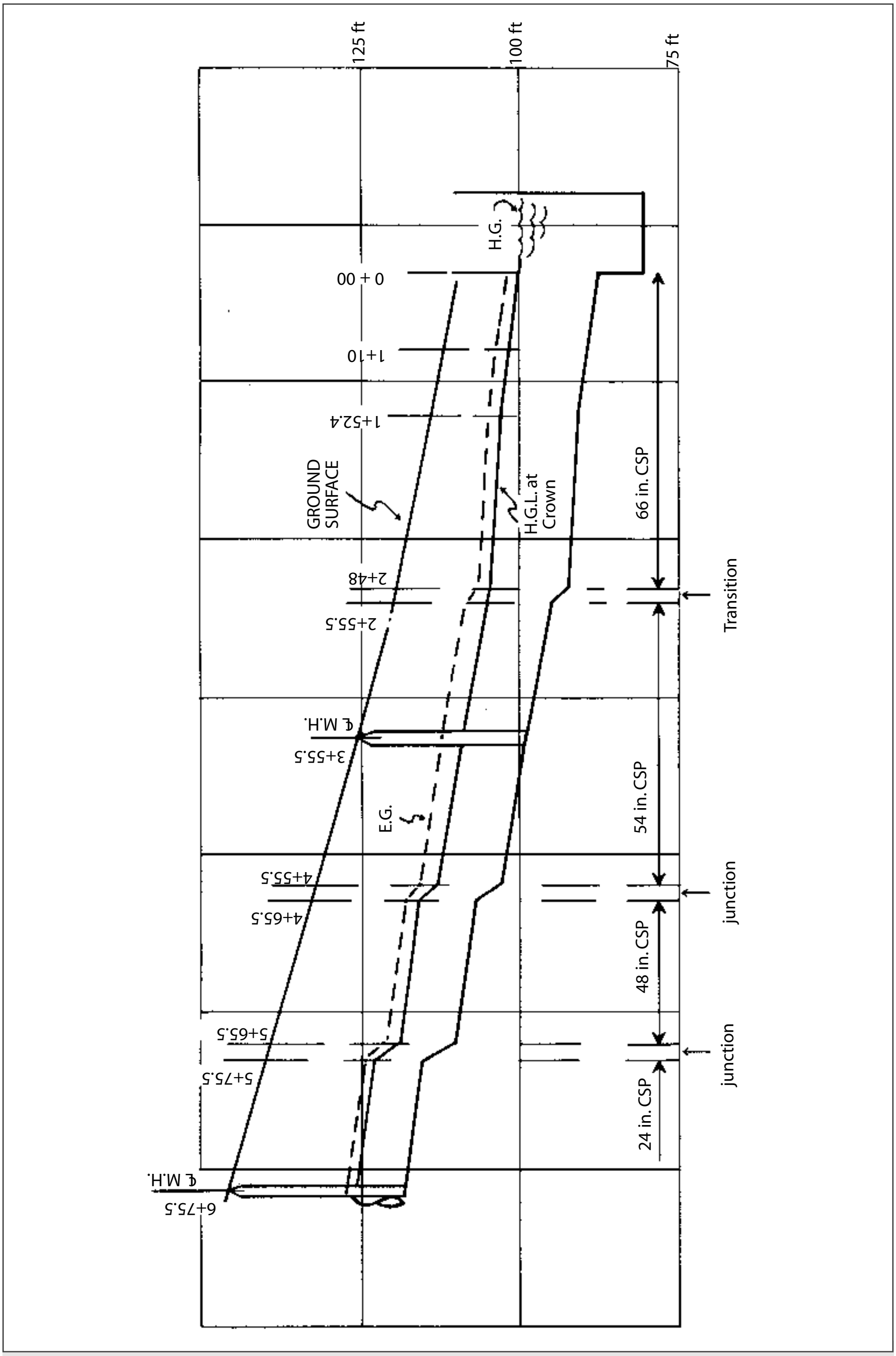
A backwater calculation will be performed in this example, using helical corrugated steel pipe, to demonstrate the significance of energy losses in sewer design. The calculation details are provided in Table 5.19.

Solution Steps:

1. Draw a plan and surface profile of the trunk storm sewer (as in Figures 5.23 and 5.24).
2. For this example, the following information is known: design discharges, Q; areas, A; and the diameters of pipes, D, have been calculated in a preliminary design.



■ **Figure 5.23** Typical plan view of storm sewer.



■ **Figure 5.24** Typical profile view of storm sewer.

Table 5.19

Sample Hydraulic Calculation Sheet

1	2	3	4	5	6		7	8	9	10	11	12	13	14	15	16	17	18	19	20
Station	Invert Elevation	Pipe Size	H.G.	Section	Area	Manning's n	K	V	Q	$V^2/2g$	E.G.	S_f	Average S_f	Length	H_f	H_b	H_j	H_m	H_t	E.G.
	(ft)	(in.)	(ft)		(ft^2)			(ft/s)	(ft^3/s)	(ft)	(ft)	(ft/ft)	(ft/ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)
0+00	94.50	66	100.00	Main	23.76	0.024	0.01680	6.10	145.00	0.58	100.58	0.0064	0.0064	110	0.70					100.58
1+10	95.20	66	100.70	Main	23.76	0.024	0.01680	6.10	145.00	0.58	101.28	0.0064	0.0064	42.4	0.27	0.08				101.28
1+52.4	95.55	66	101.05	Main	23.76	0.024	0.01680	6.10	145.00	0.58	101.63	0.0064	0.0064	95.6	0.61					101.63
2+48	96.16	66	101.66	Main	23.76	0.024	0.01680	6.10	145.00	0.58	102.24	0.0064	0.0110	7.5	0.08				0.14	102.24
2+55.5	97.38	54	101.88	Main	15.90	0.022	0.01412	9.12	145.00	1.29	103.17	0.0156	0.0156	100	1.56			0.06		103.17
3+55.5	99.00	54	103.50	Main	15.90	0.022	0.01412	9.12	145.00	1.29	104.79	0.0156	0.0156	100	1.56					104.79
4+55.5	100.56	54	105.06	Main	15.90	0.022	0.01412	9.12	145.00	1.29	106.35	0.0156	0.0135	10	0.14		0.87			106.35
4+65.5	102.07	48	106.07	Main	12.57	0.020	0.01167	7.96	100.00	0.98	107.05	0.0114	0.0114	100	1.14					107.05
5+65.5	103.21	48	107.21	Main	12.57	0.020	0.01167	7.96	100.00	0.98	108.19	0.0114	0.0117	10	0.12		3.78			108.19
5+75.5	109.11	24	111.11	Main	3.14	0.016	0.00747	6.37	20.00	0.63	111.74	0.0119	0.0119	100	1.19			0.03		111.74
6+75.5	110.33	24	112.33	Main	3.14	0.016	0.00747	6.37	20.00	0.63	112.96	0.0119								112.96

n = Variable $K = \frac{2g(n^2)}{2.21}$ $S_f = K\left(\frac{V^2}{2g}\right) + R^{4/3}$ $\Sigma H_{friction} = 7.37$ ft $\Sigma H_{energy} = 4.96$ ft

3. Perform calculations for the first section of pipe. Note that the normal depth is greater than the critical depth ($y_n > y_c$)so calculations begin at the outfall and move upstream. The design conditions at the "point of control" (outfall) are shown on the profile and calculation sheets:

Point of Control: Station 0 + 00 (outfall)

Note that the numbers in parentheses refer to the columns in Table 5.19.

Given:

Design discharge	Q	= 145 ft^3/s	(9)
Invert elevation of pipe		= 94.50 ft	(2)
Pipe diameter	D	= 66 in.	(3)
Hydraulic grade elevation	H.G.	= 100 ft	(4)
Area of pipe	A	= 23.76 ft^2	(6)
Velocity = Q/A	V	= 6.1 ft/s	(8)

Compute:

a. K (7): $K = (2g)\, n^2 / (1.486)^2$ (Derived from Manning-Chezy equation)

b. S_f (12):

$$S_f = K \frac{V^2}{2g} \div R^{4/3}$$

The friction slope (S_f) may be estimated from the relationships and formula in Table 5.20 for the expected flow, Q, and a given diameter of pipe with a known 'n' value.

S_f is a "point slope" at each station set by the designer. Therefore, the average friction slope, (Avg. S_f) (13), for each reach of pipe, L (14), is the average of the two point slopes S_f being considered.

c. Velocity Head (l0):

$$H_v = \frac{V^2}{2g}$$

d. Energy grade point, E.G. (11): H.G. (4) plus the velocity head (10)

e. Friction loss (15): H_f (15) = Avg. S_f (13) multiplied by the length of sewer section, L (14)

f. Calculate energy losses: H_b (16), H_j (17), H_m (18), H_t (19), using equations presented in this chapter (detailed calculations follow this design step summary)

g. Compute new H.G. (4) by adding all energy loss columns, (15) through (19), to the previous H.G.

h. Set new E.G. (20) equal to E.G. (11)

   Note: If the sewer system is designed to operate under pressure (surcharging), then energy losses must be added (or subtracted, depending on whether working upstream or downstream) to the energy grade line, E.G.

i. Determine pipe invert elevation (2): In this example, we are designing for full flow conditions Therefore, H.G. (4) is at the crown of the pipe and the pipe invert (2) is set by subtracting the pipe diameter, D (3), from H.G. (4).

j. Continue to follow the above procedure taking into account all form losses

k. Complete the profile drawing showing line, grade and pipe sizes. This saves time and usually helps in spotting design errors

Energy Losses Calculations (step f above)

Station 1+10 to 1+52.4 (Bend)

$$H_b = K_b\left(\frac{V^2}{2g}\right) \text{ where } K_b = 0.25\sqrt{\frac{Ø}{90}}$$

Ø, central angle of bend = 30°

$$K_b = 0.25\sqrt{\frac{30}{90}} = 0.1443$$

$$\therefore H_b = 0.1443\ (0.58) = 0.08 \text{ ft}$$

Station 2+48 to 2+55.5 (Transition)

$$H_t = 0.2\left(\frac{V_1{}^2}{2g} - \frac{V_2{}^2}{2g}\right)$$

$$= 0.2(1.29 - 0.58)$$

$$= 0.14 \text{ ft}$$

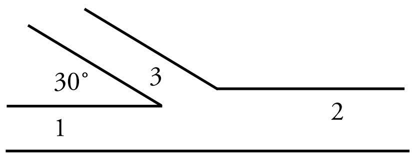
Station 3+55.5 (Manhole)

$$H_m = 0.05 \left(\frac{V^2}{2g}\right)$$

$$= 0.05\ (1.29)$$

$$= 0.06\ \text{ft}$$

Station 4+55.5 to 4+65.5 (Junction)



Q_1 = 100 ft^3/s	Q_2 = 145 ft^3/s	Q_3 = 45 ft^3/s
A_1 = 12.57 ft^2	A_2 = 15.90 ft^2	A_3 = 4.91 ft^2
D_1 = 48 in.	D_2 = 54 in.	D_3 = 30 in.
		θ_3 = 30°

$\sum P = \sum M$ (Pressure plus momentum laws)

$$H_j + D_1 - D_2 \left(\frac{A_1 + A_2}{2}\right) = \frac{Q_2^{\,2}}{A_2 g} - \frac{Q_1^{\,2}}{A_1 g} - \frac{Q_3^{\,2}\cos\theta_3}{A_3 g}$$

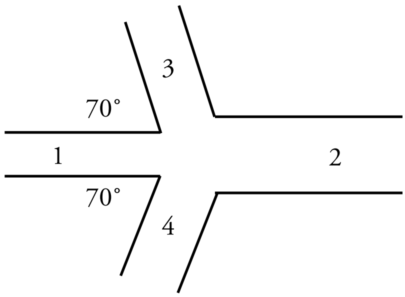
$$(H_j + 4 - 4.5)\,\frac{12.57+15.90}{2} = \frac{(145)^2}{(15.90)(32.2)} - \frac{(100)^2}{(12.57)(32.2)} - \frac{(45)^2\cos 30°}{(4.91)(32.2)}$$

$$14.24\ H_j - 0.5\ (14.24) = 41.07 - 24.71 - 11.09$$

$$14.24\ H_j - 7.12 = 5.27$$

$$H_j = 0.87\ \text{ft}$$

Station 5+65.5 to 5+75.5 (Junction)



$Q_1 = 20\ ft^3/s$ $\quad Q_2 = 100\ ft^3/s$ $\quad Q_3 = 60\ ft^3/s$ $\quad Q_4 = 20\ ft^3/s$

$A_1 = 3.14\ ft^2$ $\quad A_2 = 12.57\ ft^2$ $\quad A_3 = 7.07\ ft^2$ $\quad A_4 = 3.14\ ft^2$

$D_1 = 24$ in. $\quad D_2 = 48$ in. $\quad D_3 = 36$ in. $\quad D_4 = 24$ in

$\theta_3 = 70°$ $\quad \theta_4 = 70°$

$$H_j + D_1 - D_2 \left(\frac{A_1 + A_2}{2}\right) = \frac{Q_2^2}{A_2 g} - \frac{Q_1^2}{A_1 g} - \frac{Q_3^2 \cos\theta_3}{A_3 g} - \frac{Q_4^2 \cos\theta_4}{A_4 g}$$

$$(H_j + 2 - 4)\ \frac{3.14 + 12.57}{2} = \frac{(100)^2}{(12.57)(32.2)} - \frac{(20)^2}{(3.14)(32.2)} - \frac{(60)^2 \cos 70°}{(7.07)(32.2)} - \frac{(20)^2 \cos 70°}{(3.14)(32.2)}$$

$$7.855\ H_j - 2\ (7.855) = 24.706 - 3.956 - 5.409 - 1.353$$

$$7.855\ H_j - 15.71 = 13.988$$

$$H_j = 3.78\ \text{ft}$$

Station 6+75.5 (Manhole)

$$H_m = 0.05 \left(\frac{V^2}{2g}\right)$$

$$= 0.05\ (0.64)$$

$$= 0.03\ \text{ft}$$

Total friction loss, H_f, for the system totaled 7.37 feet. Total form energy losses for the system totaled 4.96 feet.

Table 5.20

Energy-loss Solution by Manning's Equation For Pipe Flowing Full

Diameter	Area	Hydraulic Radius			$\left(\frac{n}{1.486\,AR^{2/3}}\right)^2 \times 10^{-7}$				
	A	R	$R^{2/3}$	$AR^{2/3}$	n = 0.012	n = 0.015	n = 0.019	n = 0.021	n = 0.024
(in.)	(ft^2)	(ft)							
6	.196	0.125	0.250	0.049	271,600	424,420	681,000	831,940	1,086,350
8	.349	0.167	0.303	0.106	58,000	90,703	145,509	177,730	232,164
10	.545	0.208	0.351	0.191	17,879	27,936	44,802	54,707	71,455
12	.785	0.250	0.397	0.312	6,698	10,466	17,797	20,605	26,791
15	1.227	0.312	0.461	0.566	2,035.6	3,180.8	5,102.5	6,234.4	8,144.6
18	1.767	0.375	0.520	0.919	772.2	1,206.5	1,935.5	2,364.7	3,088.7
21	2.405	0.437	0.576	1.385	340.00	531.24	852.60	1,041.0	1,359.98
24	3.142	0.500	0.630	1.979	166.5	260.04	417.31	510.20	666.39
30	4.909	0.625	0.731	3.588	50.7	79.126	127.01	155.12	202.54
36	7.069	0.750	0.825	5.832	19.20	29.953	48.071	58.713	76.691
42	9.621	0.875	0.915	8.803	8.40	13.148	21.096	25.773	33.667
48	12.566	1.000	1.000	12.566	4.130	6.452	10.353	12.647	16.541
54	15.904	1.125	1.082	17.208	2.202	3.440	5.520	6.741	8.817
60	19.635	1.250	1.160	22.777	1.257	1.965	3.337	3.848	5.030
66	23.758	1.375	1.236	29.365	0.756	1.182	1.895	2.316	3.026
72	28.274	1.500	1.310	37.039	0.475	0.743	1.192	1.456	1.902
78	33.183	1.625	1.382	45.859	0.310	0.485	0.777	0.950	1.241
84	38.485	1.750	1.452	55.880	0.209	0.326	00.524	0.640	0.835
90	44.179	1.875	1.521	67.196	0.144	0.226	0.362	0.442	0.578
96	50.266	2.000	1.587	79.772	0.102	0.160	0.257	0.314	0.410
108	63.617	2.250	1.717	109.230	0.055	0.085	0.137	0.167	0.219
114	70.882	2.375	1.780	126.170	0.041	0.064	0.103	0.125	0.164
120	78.540	2.500	1.842	144.671	0.031	0.049	0.078	0.098	0.125

Manning Flow Equation: $Q = \left(A \times \frac{1.486}{n} \times R^{2/3}\right) \times S^{1/2}$

Energy Loss = $S = Q^2 \left(\frac{n}{1.486\,AR^{2/3}}\right)^2$

To find energy loss in pipe friction for a given Q, multiply Q^2 by the factor under the proper value of n

In this example, the head losses at the transition and junctions could also have been accommodated by either increasing the pipe diameter or increasing the slope of the pipe.

This backwater example was designed under full flow conditions but could also have been designed under pressure, allowing surcharging in the manholes, which would have reduced the pipe sizes. Storm sewer systems, in many cases, can be designed under pressure to surcharge to a tolerable hydraulic grade elevation.

Method Of Determining Equivalent Hydraulic Alternatives

A method has been developed to aid the designer in quickly determining equivalent pipe sizes for alternative materials, rather than computing the backwater profiles for each material.

The derivation shown below allows the designer to assign representative values for loss coefficients in the junctions and for average pipe length between the junctions, and develop a relationship for pipes of different roughness coefficients. As a result, the designer need only perform a detailed hydraulic analysis for one material, and then easily determine pipe sizes required for alternative materials. The relationships for hydraulic equivalent alternatives in storm sewer design are derived from the friction loss equation.

The total head loss in a sewer system is composed of junction (form) losses and friction losses:

$$H_T = H_j + H_f$$

where: H_T = Total head loss, ft
H_j = Junction losses, ft
H_f = Friction losses, ft

For the junction losses:

$$H_j = K_j \frac{V^2}{2g}$$

$$= K_j \frac{Q^2}{A^2 2g}$$

$$= K_j \frac{Q^2(16)}{\pi^2 D^4 2g}$$

For the friction losses:

$$V = \frac{(1.486)}{n} R^{2/3} S_f^{1/2}$$

$$S_f^{1/2} = \frac{n\,V}{(1.486)R^{2/3}}$$

$$S_f = \frac{n^2\,V^2}{(1.486)^2 R^{4/3}}$$

$$S_f = \frac{2g\,n^2}{(1.486)^2 R^{4/3}} \frac{V^2}{2g}$$

$$H_f = \frac{2g\;n^2\;L}{(1.486)^2 R^{4/3}} \frac{V^2}{2g}$$

$$H_f = \frac{2g\;n^2\,L\,Q^2}{(1.486)^2 R^{4/3}\,A^2 2g}$$

$$H_f = \frac{2g\;n^2\,L\,Q^2\,(16)}{(1.486)^2 R^{4/3}\,\pi^2 D^4 2g}$$

$$H_f = \frac{2g\;n^2\,L\,Q^2\,(16)\;(4)^{4/3}}{(1.486)^2 D^{4/3}\,\pi^2 D^4 2g}$$

$$H_f = \frac{2g(4)^{4/3}}{(1.486)^2} \frac{n^2\,L}{D^{4/3}} \frac{Q^2(16)}{\pi^2 D^4 2g}$$

Then, for the total head loss:

$$H_T = H_j + H_f$$

$$= K_j \frac{Q^2(16)}{\pi^2 D^4 2g} + \frac{2g(4)^{4/3}}{(1.486)^2} \frac{n^2 L}{D^{4/3}} \frac{Q^2(16)}{\pi^2 D^4 2g}$$

$$= \frac{8Q^2}{g\pi^2} \left[\frac{K_j}{D^4} + \frac{2g(4)^{4/3}}{(1.486)^2} \frac{n^2 L}{D^{16/3}} \right]$$

$$= \frac{8Q^2}{g\pi^2} \left[\frac{K_j D^{4/3} + \frac{2g(4)^{4/3}}{(1.486)^2} n^2 L}{D^{16/3}} \right]$$

Thus, for comparison of a smooth walled pipe and a corrugated steel pipe (the subscripts "s" and "c" denote smooth and corrugated):

$$\frac{8Q^2}{g\pi^2} \left[\frac{K_j D_S{}^{4/3} + \frac{2g(4)^{4/3}}{(1.486)^2} n_S{}^2 L}{D_S{}^{16/3}} \right] = \frac{8Q^2}{g\pi^2} \left[\frac{K_j D_C{}^{4/3} + \frac{2g(4)^{4/3}}{(1.486)^2} n_C{}^2 L}{D_C{}^{16/3}} \right]$$

The flow, Q, for each pipe will be the same, therefore the relationship simplifies to:

$$\frac{K_j D_S{}^{4/3} + \frac{2g(4)^{4/3}}{(1.486)^2} n_S{}^2 L}{D_S{}^{16/3}} = \frac{K_j D_C{}^{4/3} + \frac{2g(4)^{4/3}}{(1.486)^2} n_C{}^2 L}{D_C{}^{16/3}}$$

The pipe length between manholes, L, and the junction loss coefficient, K_j, may be derived from hydraulic calculations already performed for one of the materials. A required pipe diameter, for a specific Manning's n, would have resulted from those calculations. The above equation can then be solved, through trial and error, for the one remaining unknown which is the pipe diameter of the alternate material.

Another approach involves the use of representative values of the average pipe length between manholes, L, and the junction loss coefficient, K_j, derived from hydraulic calculations already performed for one of the materials. Each side of the above equation is solved independently for a number of pipe diameters and the specific Manning's n for the pipe material (and in some cases a specific Manning's n for the diameter of pipe). The results are plotted on a semi-log scale, and the plot is used to select pipe diameters of alternate materials that will provide the same performance.

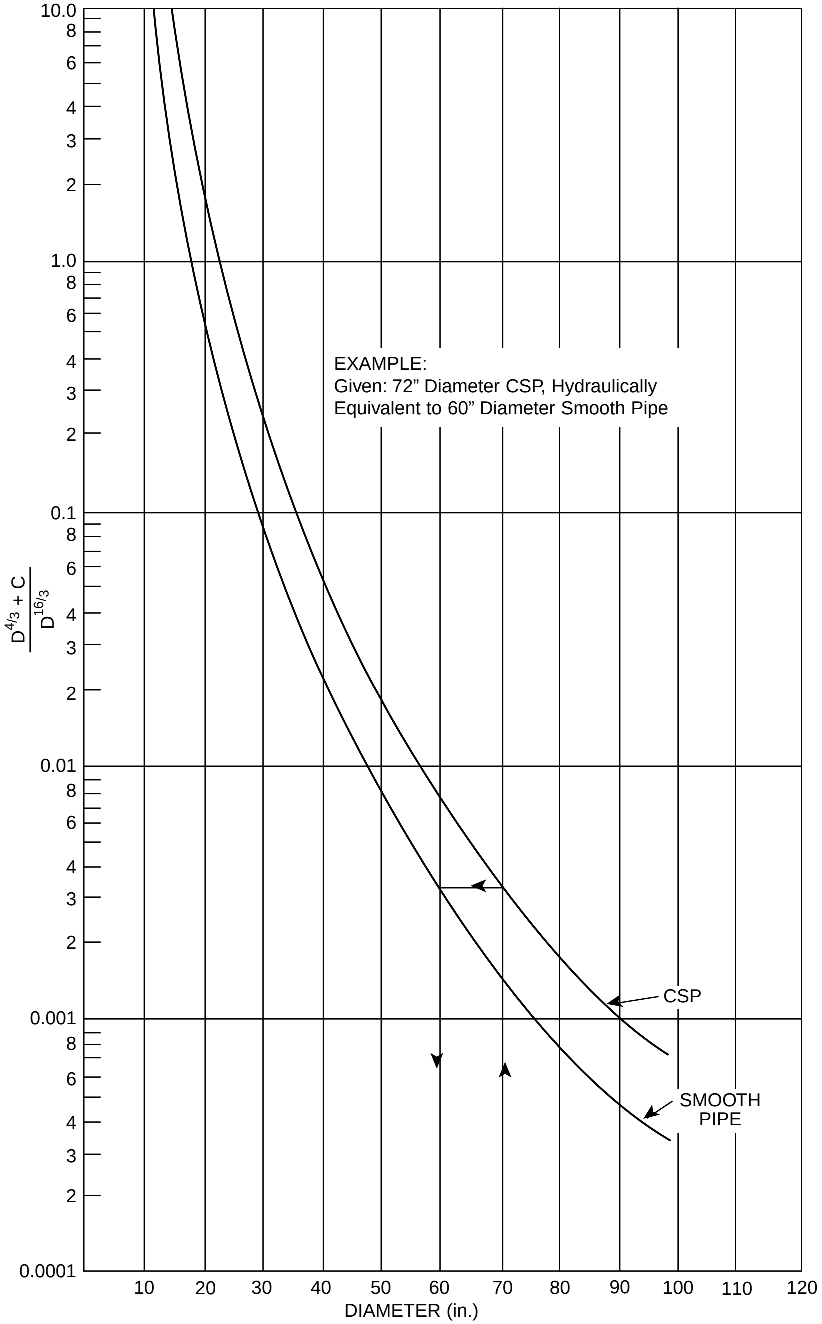
For example, Table 5.21 contains the results of such calculations based on an average pipe length of 300 feet, an average junction loss coefficient of 1.0, and Manning's n values for the pipe materials and sizes shown. The results are plotted, using a semi-log scale, so that hydraulically equivalent materials may be easily selected as demonstrated in Figures 5.25 and 5.26.

Table 5.21

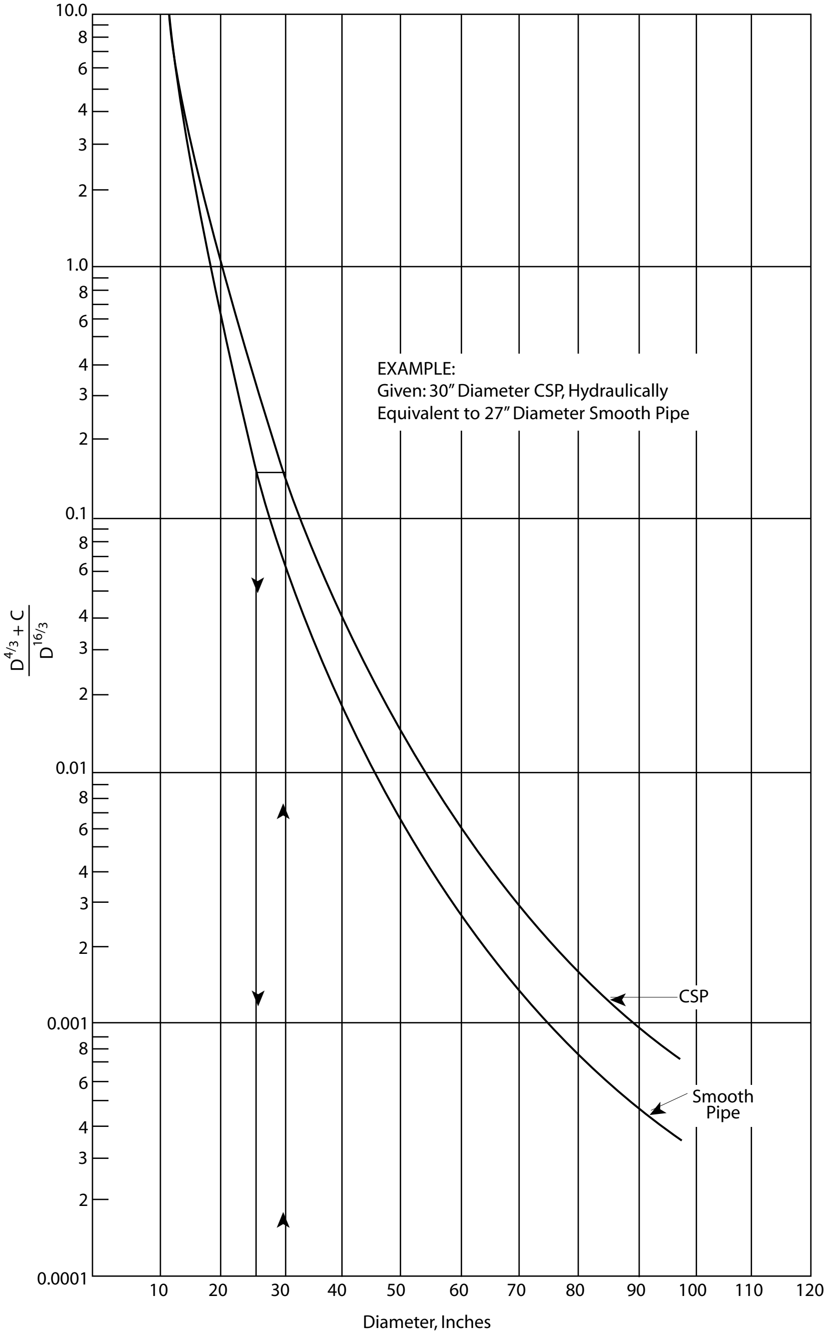
Determining Equivalent Alternatives

	Junction and Friction Losses (K_j = 1.0, L = 300 ft)			
	Smooth Pipe	Annular CSP Pipe	Helical CSP Pipe	
	n = 0.012	n = 0.024		
Diameter (in.)	$\frac{D^{4/3} + 8.00}{D^{16/3}}$	$\frac{D^{4/3} + 32.00}{D^{16/3}}$	n	$\frac{D^{4/3} + 55\,554\,n^2}{D^{16/3}}$
12	9.00	33.00	0.011	7.72
15	2.84	10.14	0.012	2.84
18	1.12	3.88	0.013	1.28
21	0.511	1.72	0.014	0.657
24	0.261	0.856	0.015	0.373
30	0.0860	0.267	0.017	0.147
36	0.0352	0.104	0.018	0.0637
42	0.0167	0.0468	0.019	0.0318
48	0.00883	0.0236	0.020	0.0176
54	0.00506	0.0129	0.021	0.0105
60	0.00310	0.00759	0.021	0.00618
66	0.00199	0.00469	0.021	0.00385
72	0.00134	0.00304	0.021	0.00251
78	0.000930	0.00204	0.021	0.00169
84	0.000665	0.00141	0.021	0.00118
90	0.000488	0.00100	0.021	0.000843
96	0.000366	0.000732	0.021	0.000618

Notes: Pipe diameter in feet in above tabular values.



■ **Figure 5.25** Equivalent alternatives to smooth wall pipe using annular CSP 2-2/3 x 1/2 in. where C = 55,554 n^2.



■ **Figure 5.26** Equivalent alternatives to smooth wall pipe using helical CSP (n variable) 2-2/3 x 1/2 in. where $C = 55{,}554\ n^2$.

Design Of Storm Drainage Facilities

System Layout

The storm drainage system layout should be made in accordance with the urban drainage objectives, following the natural topography as closely as possible. Existing natural drainage paths and watercourses, such as streams and creeks, should be incorporated into the storm drainage system. Thus the storm design should be undertaken prior to finalization of the street layout to effectively incorporate the major-minor drainage concepts.

Topographic maps, aerial photographs, and drawings of existing services are required before a thorough storm drainage design may be undertaken.

Existing outfalls, within the proposed development and adjacent lands for both the minor and major system, should be located. Allowances should be made in the design for runoff from external lands draining through the proposed development both for present conditions and future developments.

The design flows used in sizing the facilities, that will comprise the drainage network, are based on a number of assumptions. Actual flows will be different from those estimated at the design stage; the designer must not be tempted by the inherent limitations of the basic flow data to become sloppy in the hydraulic design. Also, designers should not limit their investigation to system performance under the design storm conditions, but should also assure that in cases where sewer capacities are exceeded, such incidents will not create excessive damage.

This requirement can only be practically achieved if the designer realizes that a dual drainage system exists. A drainage system is comprised of a minor system and a major system. The minor (pipe) system is typically utilized for smaller, more frequent rainfall events, while the major (overland) system is used for extreme rainfall events.

In the layout of an effective storm drainage system, the most important factor is to assure that a drainage path both for the minor and major systems be provided to avoid flooding and ponding in undesirable locations.

Minor System

The minor system consists chiefly of the storm sewer comprised of inlets, pipes, manholes and other appurtenances designed to collect and convey into a satisfactory system outfall, storm runoff for frequently occurring storms (2 to 5-year design).

Storm sewers are usually located in rights-of-way such as roadways and easements for ease of access during repair or maintenance operations.

Major System

The major drainage system will come into operation when the minor system's capacity is exceeded or when inlet capacities significantly control inflow into the minor system. In developments where the major system has been planned, the streets will act as open channels draining the excess storm water. The depth of flow on the streets should be kept within reasonable limits for reasons of safety and convenience. Consideration should be given to the area of flooding and its impact on various street classifications and to public and private property. Typical design considerations are given in Table 5.22.

Table 5.22

Typical Maximum Allowable Flow Depths

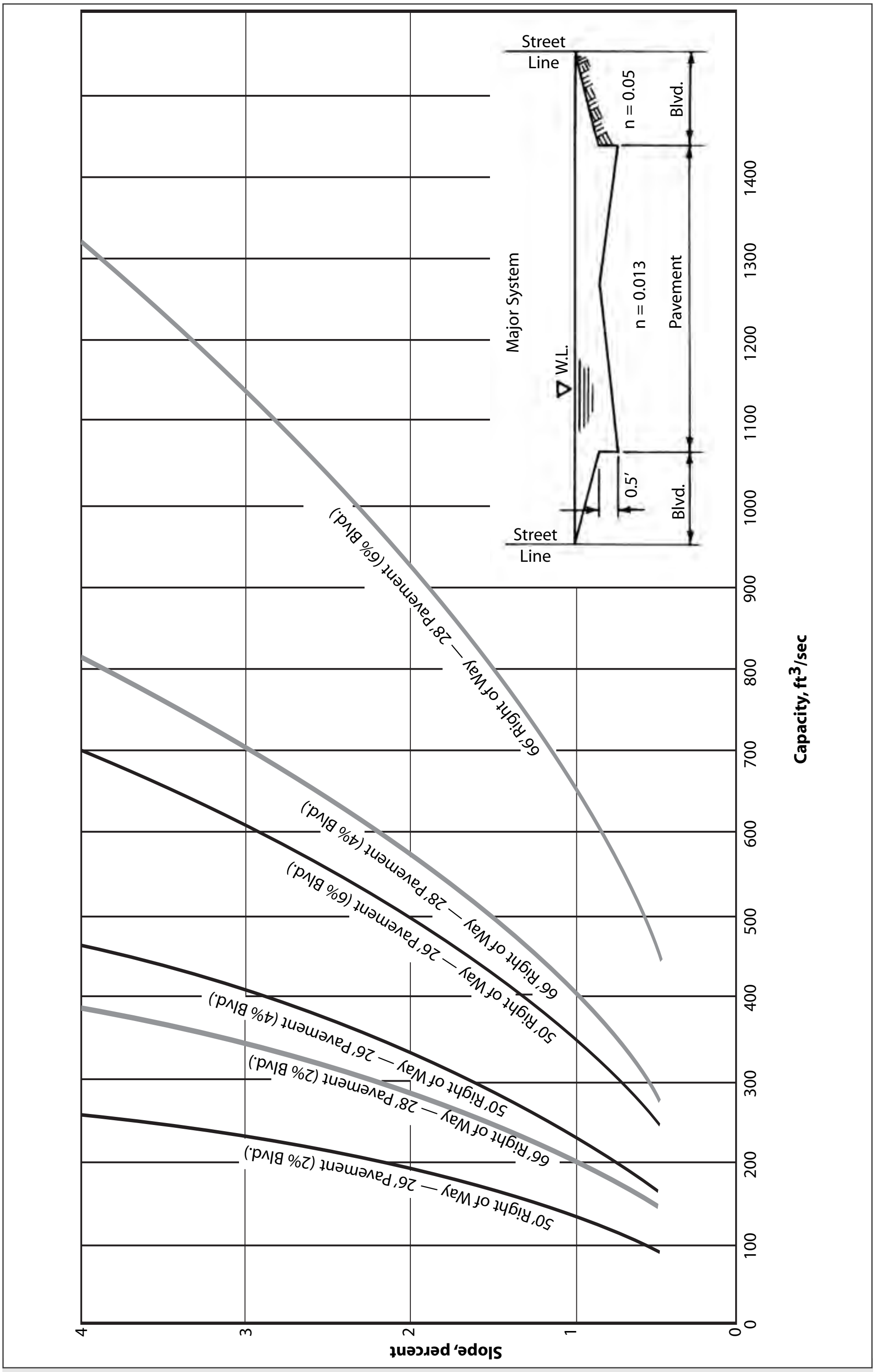
Location*	Storm Return Frequency (Years)		
	5	25	40
Walkways, Open spaces	Minor surface flow up to 1 in. deep on walkways	As required for overland flow outlets	As required for overland flow outlets
Minor, Local and Feeder Roads	3 ft wide in gutters or 4 in. deep at low point catch basins	4 in. above crown	8 in. above crown
Collector and Industrial Roads	Minor surface flow 1 in.	Up to crown	4 in. above crown
Arterial Roads	Minor Surface flow 1 in.	1 lane clear	Up to crown

Notes: *In addition to the above, residential buildings, public, commercial and industrial buildings should not be inundated at the ground line for the 100 year storm, unless buildings are flood-proofed.

To prevent the flooding of basement garages, driveways must meet or exceed the elevations corresponding to the maximum flow depth at the street.

The flow capacity of the streets may be calculated from the Manning equation, or Figure 5.27 may be used to estimate street flows.

When designing the major system, it should be done in consideration of the minor system, with the sum of their capacities being the total system's capacity. The minor system should be first designed to handle a selected high frequency storm (i.e., 2 to 5 or 10-year storm). The major system is then designed for a low frequency storm, (i.e., 100-year storm). If the roadway cannot handle the excess flow, the minor system should be enlarged to compensate.



■ **Figure 5.27** Hydraulic capacity of roadways.

HYDRAULIC DESIGN EXAMPLE OF MINOR-MAJOR SYSTEM

Description of Site

The site for this design example is shown on Figure 5.28.

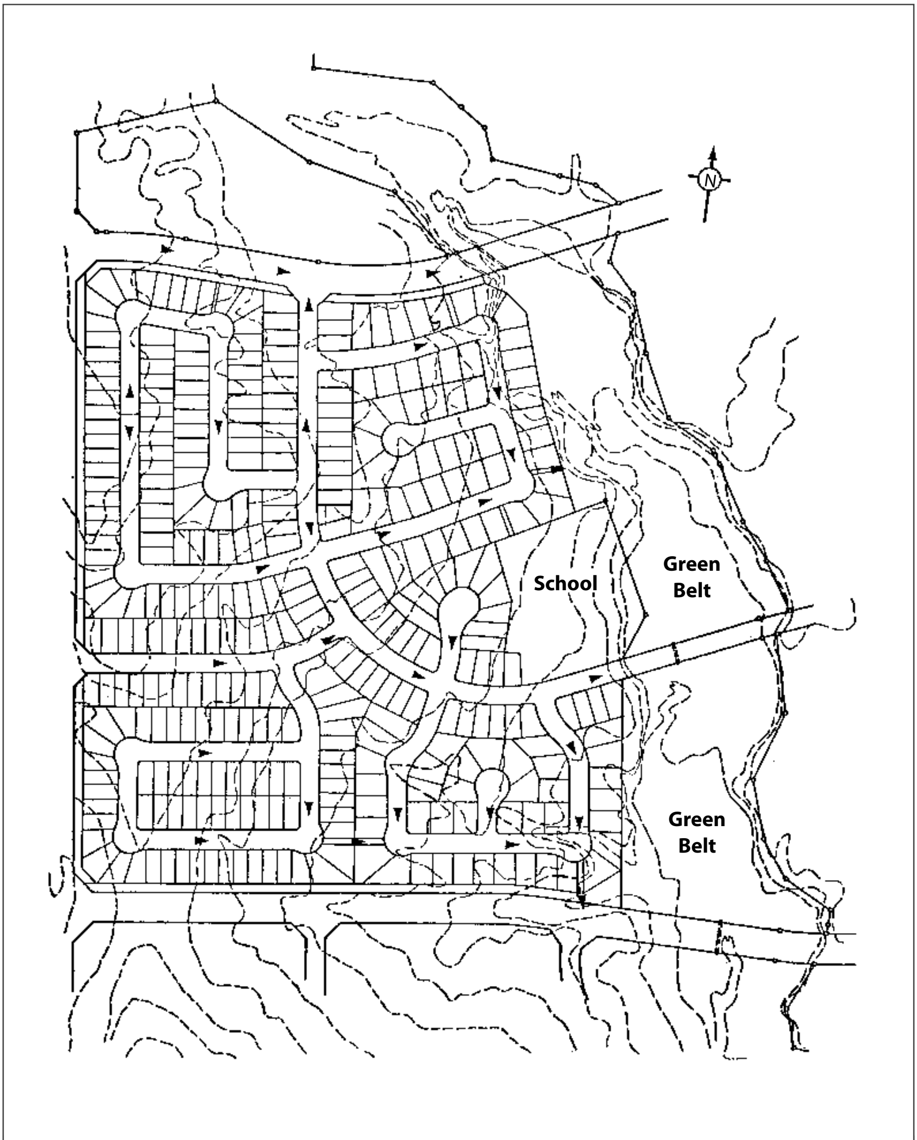
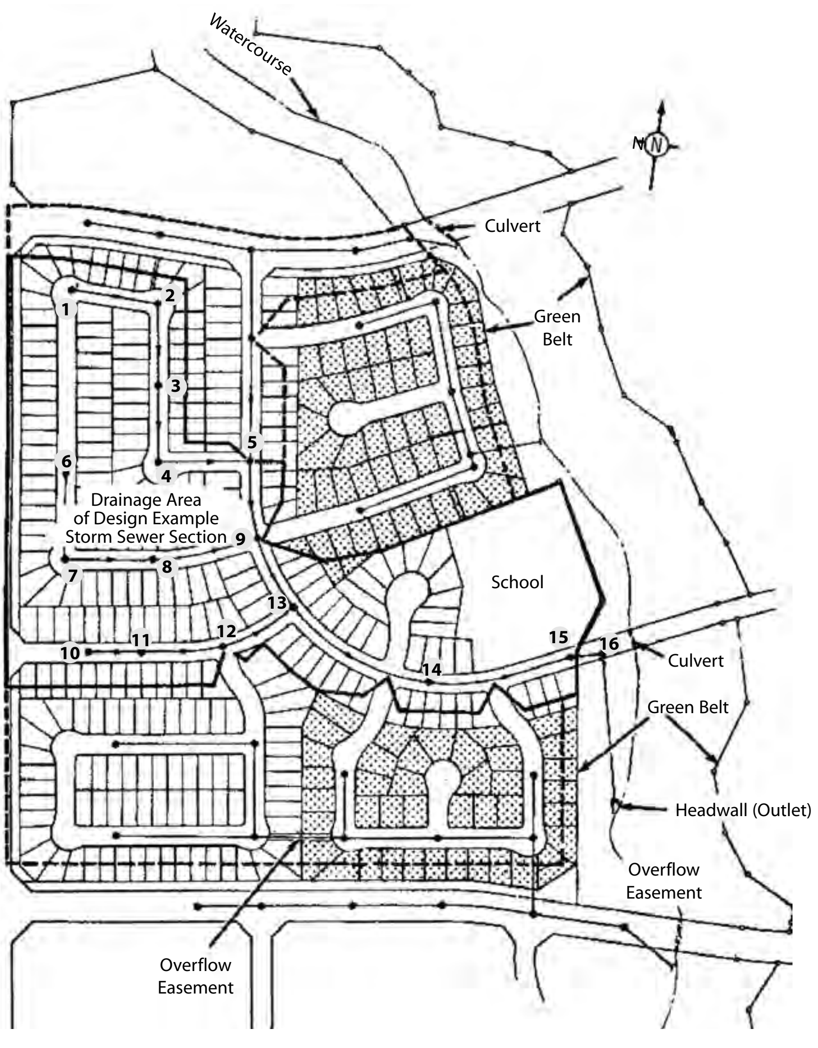


Figure 5.28 Typical site plan with route of surface runoff.

The site is about 38 acres in size consisting of single family and semi-detached housing as well as a site for a public school. The site slopes generally from west to east, where it is bounded by a major open water course. To accommodate the principles of the "minor-major" storm drainage systems, the streets have been planned to conform as much as possible to the natural contours of the lands. Where sags in roadways between intersections could not be avoided, overflow easements or walkways have been provided to permit unobstructed surface runoff during major storms, as shown on Figure 5.29.



■ **Figure 5.29** Typical site plan with storm drainage areas.

Selected Design Criteria

Based on a reasonable level of convenience to the public, a two-year storm is considered adequate as a design basis for the minor system within this development. The major (or overflow) system will be checked together with the minor system against a 100-year storm intensity. The combination of these two systems must be able to accommodate the 100-year storm runoff.

Minor System

For the limited extent of area involved, designing on the principles of the minor-major drainage concept, without gravity connections to foundation drains, permits considerable tolerance in the degree of accuracy of runoff calculations such that the rational equation is considered adequate. The values for the two-year rainfall intensity curve, obtained from local records, are shown in Table 5.23.

Table 5.23

Example Rainfall Intensity-Duration-Frequency Data

Time	2-Year Return	100 Year Return
(Min)	(in./hr)	(in./hr)
5	4.15	10.33
10	2.85	7.04
15	2.25	5.74
20	1.88	4.80
25	1.65	4.30
30	1.47	3.81
35	1.32	3.50
40	1.20	3.20
45	1.10	2.90
50	1.04	2.70
55	0.96	2.50
60	0.98	2.31
65	0.81	2.15
70	0.75	2.00
75	0.69	1.85
80	0.63	1.75
85	0.58	1.63
90	0.53	1.55
95	0.49	1.50
100	0.45	1.30
125	0.40	1.27
150	0.35	1.00
175	0.31	0.90
200	0.27	0.86

The following steps should be followed in the hydraulic design of the minor system:

1. A drainage area map is prepared indicating the drainage limits for the site, external tributary areas, the location of imported minor system and carryover flows, a proposed minor-major system layout, and the direction of surface flow.

2. The drainage area is divided into sub-areas tributary to the proposed storm sewer inlets. In this case, the inlet is located at the upstream end of each pipe segment.

3. The area of each sub-area is calculated.

4. Appropriate runoff coefficients are developed for each sub-area. The example has been simplified in that impervious areas discharging to grass areas have been given a runoff coefficient equal to that of the grassed area. The runoff coefficients chosen for this example are 0.20 for grassed areas and areas discharging to grass such as roof, patios and sidewalks), and 0.95 for impervious surfaces (streets and driveways). This results in an average runoff coefficient of 0.35 for all the sub-areas of this specific site. The area of each sub-area is multiplied by the runoff coefficient for that sub-area, and summed for each section of sewer. An "area x C" value is determined for each inlet.

5. The inflow rate for the first inlet of a sewer section is calculated by the rational method, using the initial time of concentration and the corresponding intensity. The inflow rate is the "area x C" value multiplied by the intensity. As mentioned above, inlets will be located at the upstream manhole for each length of pipe. In this example, the initial values are as flows:

   T_c = Time of concentration = 10 minutes.
   i = 2.85 in./hr for the 2-year storm (Table 5.23).

6. Commencing at the upstream end of the system, the inflow to be carried by each pipe is calculated one at a time. The initial time of concentration, as mentioned above, is 10 minutes at the most up-stream inlet of each section of sewer. In this design example, a helical 2 2/3 x 1/2 in. corrugation CSP with variable roughness coefficients (Table 5.9) has been selected as the pipe material. The capacity of a pipe is calculated using Manning's n, the slope of the pipe and the section properties of the pipe flowing full ($AR^{2/3}$). Once a tentative pipe size is chosen that will accommodate the required flow rate for the first pipe, the flow velocity is calculated (flow rate divided by pipe end area), the travel time is determined (pipe length divided by velocity), and the new time of concentration for the next pipe is the sum of the previous time of concentration and the travel time. The resulting time of concentration is then used to determine a new intensity for the next inflow (required capacity) calculation. At a junction of two or more pipes, the longest time of concentration is selected and the procedure continues downstream. If upon completion of the hydraulic design (and backwater calculations) the times of concentrations have varied enough to alter the discharges, new flow values should be determined. In most cases the slight variance in the time of concentration will not significantly affect the peak flows.
   Note that design velocities in storm sewers should be a minimum of 3 ft/s when

flowing half full to full to prevent deposition by attaining self cleaning velocities. A maximum allowable velocity of 15 ft/s is used so as to avoid erosive damage to the pipe.

The above computations are summarized in Table 5.24.

7. Manhole and junction losses are considered as the design proceeds downstream. Certain rules of thumb may be used in the preliminary hydraulic analysis. In this design example, the following manhole drops are assumed:

   0.05 feet for straight runs
   0.15 feet for junctions with up to a 45° change in direction
   0.25 feet for 45° to 90° junctions

   Also, crowns of incoming and outgoing pipes at manholes are kept equal where the increase in downstream diameter meets or exceeds the above manhole drops.

   The preliminary minor system design is shown in Table 5.24 with the tentative pipe sizes and manhole drops.

8. The hydraulic analysis is then performed on the proposed minor system to ensure that it operates as expected. The hydraulic grade is set at the crown of the outlet pipe, with hydraulic calculations proceeding upstream. The energy loss equations are done using the same procedure as previously detailed. The detailed hydraulic calculations follow this step-by-step design summary, with the results summarized in Table 5.25.

   In this example the initial pipe sizes did not change, but rather manhole drops were adjusted to account for the junction losses. If junction losses had resulted in the elevation of the pipe crown exceeding the minimum cover criteria, then the hydraulic grade line may have been lowered by increasing the pipe size. The hydraulic grade line may be permitted to exceed the crown where surcharging in the storm system can be tolerated.

9. The designer can estimate the required pipe sizes for a minor system for an alternative pipe material or roughness coefficient. There is no need to perform a detailed hydraulic analysis for the alternative pipe, but rather use the method of "Equivalent Alternatives" as described earlier in this chapter. In this example, the average length of pipe is estimated to be 300 feet with an average manhole junction loss coefficient of 1.0. The alternative pipe will have constant n = .012. The results are summarized in Table 5.26.

Table 5.24

Example Preliminary Storm Sewer Design

Sewer Description and Data									Runoff Calculation							Pipe Design							Time of Concentration	
							M.H. Invert El.																Entry = 10 min.	
Street	From M.H.	To M.H.	Length	Slope	Fall	M.H. Drop	Up-Stream	Down-Stream	Drainage Area A	C	A x C	Total Section A x C	Total Trunk A x C	Intensity I	Flow Q	Pipe Size	Area A	Pipe $AR^{2/3}$	Manning's n	Actual Cap. Q	Velocity V	Sect.ion	Accum.
			(ft)	%	(ft)	(ft)	(ft)	(ft)	(ac)		(ac)	(ac)	(ac)	(in./hr)	(ft^3/s)	(in.)	(ft^2)	($ft^{8/3}$)		(ft^3/s)	(ft/s)	(min.)	(min.)
A	1	2	300	0.84	2.52		771.71	769.19	1.82	0.35	0.64	0.64		2.85	1.82	10	0.545	0.192	0.014	1.87	3.43	1.46	11.46
A	2	3	260	1.30	3.38	0.25	768.94	765.56	2.73	0.35	0.96	1.60		2.67	4.27	12	0.785	0.312	0.012	4.41	5.62	0.77	12.23
A	3	4	265	0.98	2.60	0.25	765.31	762.71	2.57	0.35	0.90	2.50		2.58	6.45	15	1.227	0.565	0.013	6.39	5.21	0.85	13.08
A	4	5	306	1.50	4.59	0.25	762.46	757.87	2.06	0.35	0.72	3.22		2.48	7.99	15	1.227	0.565	0.013	7.91	6.45	0.79	13.87
B	6	7	300	1.70	5.10		771.62	766.52	2.63	0.35	0.92	0.92		2.85	2.62	10	0.545	0.192	0.014	2.66	4.88	1.02	11.02
B	7	8	300	1.70	5.10	0.25	766.27	761.17				0.92			2.62	10	0.545	0.192	0.014	2.66	4.883	1.02	12.04
B	8	9	245	2.20	5.39	0.17	761.00	755.61	3.70	0.35	1.30	2.22		2.61	5.79	12	0.785	0.312	0.012	5.73	7.30	0.56	12.604
C	10	11	300	1.40	4.20		767.54	763.34	4.46	0.35	1.56	1.56		2.85	4.45	12	0.785	0.312	0.012	4.57	5.82	0.86	10.86
C	11	12	275	2.40	6.60	0.05	763.29	756.69	1.76	0.35	0.62	2.18		2.75	6.00	12	0.785	0.312	0.012	5.99	7.63	0.60	11.46
C	12	13	265	1.10	2.92	0.25	756.44	753.52	1.05	0.35	0.37	2.55		2.67	6.81	15	1.227	0.565	0.013	6.77	5.52	0.80	12.26
Main	5	9	265	0.76	2.01	0.25	757.62	755.61	1.06	0.35	0.37		3.59	2.39	8.58	18	1.767	0.919	0.014	8.50	4.81	0.92	14.79
Main	9	13	265	0.60	1.59	0.50	755.11	753.52	1.32	0.35	0.46		6.27	2.28	14.30	24	3.142	1.979	0.016	14.24	4.53	0.97	15.76
Main	13	14	500	1.70	8.50	0.25	753.27	744.77	5.64	0.35	1.97		10.79	2.19	23.63	24	3.142	1.979	0.016	23.96	7.63	1.09	16.85
Main	14	15	500	1.80	9.00	0.05	744.72	735.72	1.37	0.35	0.48		11.27	2.11	23.78	24	3.142	1.979	0.016	24.66	7.854	1.06	17.91
Main	15	16	110	1.20	1.32	0.25	735.47	734.15	5.81	0.20	1.16		12.43	2.03	25.23	27	3.976	2.709	0.017	25.94	6.52	0.28	18.19
Main	16	Outfall	500	.068	3.40	0.25	733.90	730.50							25.23	30	4.909	3.589	0.017	25.87	5.27	1.58	19.77

Table 5.25

Example Hydraulic Calculation Sheet

M.H.	Invert Elevation	Pipe Size	H.G.	Manning's n	Area	K	Q	V	$\frac{V^2}{2g}$	E.G.	S_f	Length	H_f	H_b	H_j	H_m	H_t	E.G.
	(ft)	(in.)	(ft)		(ft^2)		(ft^3/s)	(ft/s)	(ft)	(ft)		(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	
Outlet	730.50	30	733.00	0.017	4.909	0.0084	25.23	5.14	0.41	733.41								733.41
16	733.86	30	736.36	0.017	4.909	0.0084	25.23	5.14	0.41	736.77	0.0064	500	3.20	0.12				736.77
15	735.41	27	737.66	0.017	3.976	0.0084	25.23	6.35	0.63	738.29	0.114	110	1.25				0.04	738.29
14	744.18	24	746.18	0.016	3.142	0.0075	23.78	7.57	0.89	747.07	0.0168	500	8.40	0.07		0.05	0.05	747.07
13	753.69	24	755.69	0.016	3.142	0.0075	23.63	7.52	0.88	756.57	0.0166	500	8.30	0.10	1.11			756.57
9	756.11	24	758.11	0.016	3.142	0.0075	14.30	4.55	0.32	758.43	0.0060	265	1.59	0.03	0.80			758.43
5	759.18	18	760.68	0.014	1.767	0.0057	8.58	4.86	0.37	761.05	0.0078	265	2.07	0.43			0.07	761.05
4	764.99	15	766.24	0.013	1.227	0.0049	7.99	6.51	0.66	766.90	0.0153	306	4.68	0.88				766.90
3	767.64	15	768.89	0.013	1.227	0.0049	6.45	5.26	0.43	769.32	0.0099	265	2.62			0.02	0.01	769.32
2	771.41	12	772.41	0.012	0.785	0.0042	4.27	5.44	0.46	772.87	0.0123	260	3.20	0.29			0.03	772.87
1	774.09	10	774.92	0.014	0.545	0.0057	1.82	3.34	0.17	775.09	0.0078	300	2.34			0.17		775.09
12	757.51	15	758.76	0.013	1.227	0.0049	6.81	5.55	0.48	759.24	0.0111	265	2.94	0.04			0.09	759.24
11	764.49	12	765.49	0.012	0.785	0.0042	6.00	7.64	0.91	766.40	0.0243	275	6.68			0.05		766.40
10	768.98	12	769.98	0.012	0.785	0.0042	4.45	5.67	0.50	770.48	0.0133	300	3.99			0.50		770.48
8	762.82	12	763.82	0.012	0.785	0.0042	5.79	7.38	0.85	764.67	0.0227	245	5.56	0.10			0.05	764.67
7	768.45	10	769.28	0.014	0.545	0.0057	2.62	4.81	0.36	769.64	0.0166	300	4.98	0.48				769.64
6	773.79	10	774.62	0.014	0.545	0.0057	2.62	4.81	0.36	774.98	0.0166	300	4.98			0.36		774.98

Table 5.26

Example Equivalent Alternatives

	Location		Pipe Size	
	M.H.	M.H.	Design	Equivalent
Street	From	To	(in.)	(in.)
A	1	2	10	10
A	2	3	12	12
A	3	4	15	15
A	4	5	15	15
B	6	7	10	10
B	7	8	10	10
B	8	9	12	12
C	10	11	12	12
C	11	12	12	12
C	12	13	15	15
Main	5	9	18	18
Main	9	13	24	24
Main	13	14	24	24
Main	14	15	24	24
Main	15	16	27	24
Main	16	Outfall	30	27

Form Loss Calculations for Minor System Design

M.H. 16 θ = 45° (Bend at manhole, no special shaping)
From Figure 5.14, $K_b = 0.3$

$$H_b = K_b\left(\frac{V^2}{2g}\right) = 0.3 \times 0.41 = 0.12 \text{ ft}$$

$$H_t = 0.2\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) = 0.2 \times (0.63\text{-}0.41) = 0.04 \text{ ft}$$

M.H. 15

$$H_t = 0.2\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) = 0.2 \times (0.89\text{-}0.63) = 0.05 \text{ ft}$$

M.H. 14 θ = 10° (Curved sewer)

$$K_b = 0.25\sqrt{\frac{\Delta}{90}} = 0.25 \times \left(\frac{10}{90}\right)^{1/2} = 0.083$$

$$H_b \;=\; K_b\left(\frac{V^2}{2g}\right) \;= 0.083 \text{ x } 0.89 = 0.07 \text{ ft}$$

$$H_m \;= 0.05\left(\frac{V^2}{2g}\right) \;= 0.05 \text{ x } 0.89 = 0.05 \text{ ft}$$

M.H. 13 $\Delta = 20°$ (Curved sewer)

$$K_b \;=\; 0.25\sqrt{\frac{\Delta}{90}} \;= 0.25 \text{ x}\left(\frac{20}{90}\right)^{1/2} = 0.118$$

$$H_b \;=\; K_b\left(\frac{V^2}{2g}\right) \;= 0.118 \text{ x } 0.88 = 0.10 \text{ ft}$$

$$(H_j + D_1 - D_2)\left(\frac{A_1 + A_2}{2}\right) = \frac{Q_2{}^2}{A_2 g} - \frac{Q_1{}^2}{A_1 g} - \frac{Q_3{}^2}{A_3 g}\;\cos\theta$$

$\theta = 90°$, $\cos 90° = 0$

$$(H_j + 2.0 - 2.0)\left(\frac{3.142 + 3.142}{2}\right) = \frac{(23.63)^2}{(3.142)(32.2)} - \frac{(14.30)^2}{(3.142)(32.2)} - 0$$

$H_j \;= 1.11$ ft

M.H. 9 $\Delta = 10°$ (Curved sewer)

$$K_b \;=\; 0.25\sqrt{\frac{\Delta}{90}} \;= 0.25 \text{ x}\left(\frac{20}{90}\right)^{1/2} = 0.118$$

$$H_b \;=\; K_b\left(\frac{V^2}{2g}\right) \;= 0.118 \text{ x } 0.88 = 0.10 \text{ ft}$$

$$(H_j + D_1 - D_2)\left(\frac{A_1 + A_2}{2}\right) = \frac{Q_2{}^2}{A_2 g} - \frac{Q_1{}^2}{A_1 g} - \frac{Q_3{}^2}{A_3 g}\;\cos\theta$$

$\theta = 90°, \cos 90° = 0$

$$(H_j + 1.5 - 2.0)\left(\frac{1.767 + 3.142}{2}\right) = \frac{(14.30)^2}{(3.142)(32.2)} - \frac{(8.50)^2}{(1.767)(32.2)} - 0$$

$H_j \;=\; 0.80$ ft

M.H. 5 $\theta = 90°$ (Bend at manhole, no special shaping)
From Figure 5.14, $K_b = 1.33$

$$H_b = K_b\left(\frac{V^2}{2g}\right) = 1.33 \text{ x } 0.32 = 0.43 \text{ ft}$$

$$H_t = 0.2\left(\frac{V_1^{\,2}}{2g} - \frac{V_2^{\,2}}{2g}\right) = 0.2 \text{ x } (0.66\text{-}0.32) = 0.07 \text{ ft}$$

M.H. 4 $\theta = 90°$ (Bend at manhole, no special shaping)
From Figure 5.14, $K_b = 1.33$

$$H_b = K_b\left(\frac{V^2}{2g}\right) = 1.33 \text{ x } 0.66 = 0.88 \text{ ft}$$

M.H. 3

$$H_t = 0.2\left(\frac{V_1^{\,2}}{2g} - \frac{V_2^{\,2}}{2g}\right) = 0.2 \text{ x } (0.46\text{-}0.43) = 0.01 \text{ ft}$$

$$H_m = 0.05\left(\frac{V^2}{2g}\right) = 0.05 \text{ x } 0.43 = 0.02 \text{ ft}$$

M.H. 2 $\theta = 60°$ (Bend at manhole, no special shaping)
From Figure 5.14, $K_b = 0.63$

$$H_b = K_b\left(\frac{V^2}{2g}\right) = 0.63 \text{ x } 0.46 = 0.29 \text{ ft}$$

$$H_t = 0.1\left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right) = 0.1 \text{ x } (0.46\text{-}0.17) = 0.03 \text{ ft}$$

M.H. 1

$$H_{tm} = \left(\frac{V^2}{2g}\right) = 0.17 \text{ ft}$$

M.H. 12 $\Delta = 10°$ (Curved sewer)

$$K_b = 0.25\sqrt{\frac{\Delta}{90}} = 0.25 \text{ x}\left(\frac{10}{90}\right)^{1/2} = 0.083$$

$$H_b = K_b\left(\frac{V^2}{2g}\right) = 0.083 \text{ x } 0.48 = 0.04 \text{ ft}$$

$$H_t = 0.2\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) = 0.2 \text{ x } (0.91\text{-}0.48) = 0.09 \text{ ft}$$

M.H. 11

$$H_m = 0.05\left(\frac{V^2}{2g}\right) = 0.05 \text{ x } 0.91 = 0.05 \text{ ft}$$

M.H. 10

$$H_{tm} = \left(\frac{V^2}{2g}\right) = 0.50 \text{ ft}$$

M.H. 8 $\Delta = 20°$ (Curved sewer)

$$K_b = 0.25\sqrt{\frac{\Delta}{90}} = 0.25 \text{ x}\left(\frac{20}{90}\right)^{1/2} = 0.118$$

$$H_b = K_b \left(\frac{V^2}{2g} \right) = 0.118 \times 0.85 = 0.10 \text{ ft}$$

$$H_t = 0.1 \left(\frac{V_2^{\,2}}{2g} - \frac{V_1^{\,2}}{2g} \right) = 0.1 \times (0.85\text{-}0.36) = 0.05 \text{ ft}$$

M.H. 7 $\theta = 90°$ (Bend at manhole, no special shaping)
From Figure 5.14, $K_b = 1.33$

$$H_b = K_b \left(\frac{V^2}{2g} \right) = 1.33 \times 0.36 = 0.48 \text{ ft}$$

M.H. 10

$$H_{tm} = \left(\frac{V^2}{2g} \right) = 0.36 \text{ ft}$$

Major System

Various manual methods can be used to estimate the major system flows. As a preliminary estimate, designers often apply the Manning equation, using the rainfall intensity for a 100-year storm and a C factor 60 to 85% higher than what would be used for a 2 or 5-year storm. The increase in value is basically to allow for a change in the antecedent moisture condition; with such large flows, there will be less infiltration and more runoff. Except in special circumstances, a C factor above 0.85 need not be used.

In this design example, the C factor of 0.35 used for the design of the minor system will be increased to 0.60, an increase of about 70 %. The results are shown in Table 5.27.

In cases where this method results in flows in excess of the acceptable roadway capacity, a more detailed method should be applied, such as the SCS Graphical Method or a suitable hydrological computer model.

If properly laid out, the major system can tolerate the variability in flows estimated by the various methods. A minor increase in the depth of surface flow will greatly increase the capacity of the major system, without necessarily causing serious flooding. The designer must also consider the remaining overland flow accumulated at the downstream end of the development. Adequate consideration must be given for its conveyance to the receiving water body. This may involve increasing the minor system and inlet capacities or providing adequate drainage swales.

Table 5.27

Example Major System Flows For 100-Year Storm

Sewer Description and Data									Runoff Calculation							Pipe Design						Time of Concentration		Major System		
							M.H. Invert El.															Entry = 10 min.				
Street	From M.H.	To M.H.	Length	Slope	Fall	M.H. Drop	Up-Stream	Down-Stream	Drainage Area A	C	A x C	Total Section A x C	Total Trunk A x C	Intensity I	Flow Q	Pipe Size	Area A	Pipe $AR^{2/3}$	Manning's n	Actual Cap. Q	Velocity V	Sect.ion	Accum.	Flow Q	Road Grade	Surface Capacity
			(ft)	%	(ft)	(ft)	(ft)	(ft)	(ac)		(ac)	(ac)	(ac)	(in./hr)	(ft^3/s)	(in.)	(ft^2)	($ft^{8/3}$)		(ft^3/s)	(ft/s)	(min.)	(min.)	(ft^3/s)	(%)	(ft^3/s)
A	1	2	300	0.89	2.68		774.09	771.41	1.82	0.60	1.09	1.09		7.04	7.67	10	0.545	0.192	0.014	1.93	3.54	1.41	11.41	5.74	2.00	182.0
A	2	3	260	1.35	3.52	0.25	771.16	767.64	2.73	0.60	1.64	2.73		6.67	18.21	12	0.785	0.312	0.011	4.90	6.24	0.69	12.10	13.31	2.00	182.0
A	3	4	265	0.91	2.40	0.25	767.39	764.99	2.57	0.60	1.54	4.27		6.49	27.71	15	1.227	0.565	0.012	6.66	5.43	0.81	12.91	21.05	2.00	182.0
A	4	5	306	1.820	5.56	0.25	764.74	759.18	2.06	0.60	1.24	5.51		6.28	34.60	15	1.227	0.565	0.012	9.43	7.69	0.66	13.57	25.17	1.90	180.0
B	6	7	300	1.78	5.34		773.79	768.45	2.63	0.60	1.58	1.58		7.04	11.12	10	0.545	0.192	0.014	2.72	4.99	1.00	11.00	8.40	2.00	182.0
B	7	8	300	1.79	5.38	0.25	768.20	762.82				1.58			11.12	10	0.545	0.192	0.014	2.73	5.01	1.00	12.00	8.39	2.20	191.0
B	8	9	245	2.67	6.54	0.17	762.65	756.11	3.70	0.60	2.22	3.80		6.52	24.78	12	0.785	0.312	0.011	6.89	8.78	0.47	12.47	17.89	2.00	182.0
C	10	11	300	1.50	4.49		768.98	764.49	4.46	0.60	2.68	2.68		7.04	18.87	12	0.785	0.312	0.011	5.16	6.57	0.76	10.76	13.71	1.85	175.0
C	11	12	275	2.52	6.93	0.05	764.44	757.51	1.76	0.60	1.06	3.74		6.84	25.58	12	0.785	0.312	0.011	6.69	8.52	0.54	11.30	18.89	2.00	182.0
C	12	13	265	1.35	3.57	0.25	757.26	753.69	1.05	0.60	0.63	4.37		6.70	29.28	15	1.227	0.565	0.012	8.12	6.62	0.67	11.97	21.16	2.20	191.0
Main	5	9	265	1.06	2.82	0.25	758.93	756.11	1.06	0.60	0.64		6.15	6.11	37.58	18	1.767	0.919	0.013	10.84	6.13	0.72	14.29	26.74	2.00	275.0
Main	9	13	265	0.72	1.92	0.50	755.61	753.69	1.32	0.60	0.79		10.74	5.92	63.58	24	3.142	1.979	0.015	16.69	5.31	0.83	15.12	46.89	2.00	275.0
Main	13	14	500	1.85	9.26	0.25	753.44	744.18	5.64	0.60	3.38		18.49	5.72	105.78	24	3.142	1.979	0.015	26.68	8.49	0.98	16.10	79.08	2.50	310.0
Main	14	15	500	1.74	8.72	0.05	744.13	735.41	1.37	0.60	0.82		19.31	5.53	106.78	24	3.142	1.979	0.015	25.89	8.24	1.01	17.11	80.89	2.00	275.0
Main	15	16	110	1.18	1.30	0.25	735.16	733.86	5.81	0.60	3.49		22.80	5.34	121.75	27	3.976	2.709	0.016	27.35	6.88	0.27	17.38	94.40	0.50	140.0
Main	16	Outfall	500	.0622	3.11	0.25	733.61	730.50							121.75	30	4.909	3.589	0.017	24.74	5.04	1.65	19.03	97.01	0.50	140.0

Foundation Drains

To establish the groundwater level, piezometer measurements over a 12 month period were taken, indicating the groundwater table would be safely below the footing elevations for the proposed buildings, minimizing the amount of inflow that can be expected into the foundation drains.

The municipal requirements include detailed lot grading control, thus further reducing the possibility of surface water entering the foundation drains. A flow value of 0.0027 ft^3/s per basement is used. Detailed calculations are provided in Table 5.28.

Computer Models

There is a wide range of computer models now available for analyzing sewer networks. The complexity of the models varies from straightforward models that use the rational method to estimate the peak flow, to comprehensive models that are based on the continuity and momentum equations. The latter are capable of modeling surcharge, backwater, orifices, weirs and other sewer components. Table 5.29 lists several of these models and their capabilities.

Table 5.28

Example Foundation Drain Collector Design Sheet

Location	From M.H.	To M.H.	Unit Area	Density	Total Units	Cum. Units	Flow Per Unit	Total Flow	Gradient	Pipe Size	Capacity	Velocity
			(acres)	(per acre)			(ft^3/s)	(ft^3/s)	(%)	(in.)	(ft^3/s)	(ft/s)
Cres.'G'	1A	2A	2.97	6	18	18	0.0027	0.049	0.98	8	1.30	3.7
'G'	2A	3A	1.78	6	11	29	0.0027	0.078	1.51	8	1.61	4.6
'G'	3A	4A	3.68	6	22	51	0.0027	0.138	0.50	8	0.93	2.7
'G'	4A	5A	1.48	6	9	60	0.0027	0.162	0.55	8	0.97	2.8
'G'	1A	6A	3.75	6	23	23	0.0027	0.062	1.39	8	1.54	4.4
'G'	6A	7A	2.30	6	14	37	0.0027	0.023	2.25	8	1.96	5.6
'G'	7A	8A	1.43	6	9	46	0.0027	0.124	1.31	8	1.50	4.3
Street 'F'	9A	10A	3.80	8	30	30	0.0027	0.081	1.20	8	1.43	4.1
'F'	10A	11A	2.10	8	17	47	0.0027	0.127	1.20	8	1.43	4.1
Street 'A'	5A	8A	1.56	8	12	72	0.0027	0.194	1.81	8	1.76	5.0
'A'	8A	11A	1.27	8	10	128	0.0027	0.346	4.34	8	2.73	7.8
'A'	11A	13A	2.33	8	19	194	0.0027	0.524	1.42	8	1.56	4.5

Table 5.29

Computer Models - Sewer System Design and Analysis

Model Characteristics	CE Storm	HVM Dorsch3	ILLUDAS	SWMM-Extran	SWMM-Transport	WASSP-SIM	WSPRO
Model Purpose:							
Hydraulic Design	●		●		●	●	●
Evaluation/Prediction		●		●	●	●	●
Model Capabilities:							
Pipe Sizing	●		●		●	●	
Weirs/Overflows		●		●	●	●	
Surcharging		●		●		●	
Pumping Stations				●	●	●	
Storage				●	●	●	
Open Channel Water Surface Profile							●
Hydraulic Equations:							
Linear Kinematic Wave	●		●				
Non-Linear Kinematic Wave					●	●	
St. Venant's - Explicit				●			
St. Venant's - Implicit		●					
Ease of Use:							
High	●		●				●
Low		●		●	●	●	

Multiple opening installation.

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CSP underground detention system being installed.

CHAPTER six

Stormwater Management and Water Quality Design

OVERVIEW

Changes in land use require designers to give careful consideration to stormwater management issues. The development of previously undisturbed areas or changes in land usage and drainage patterns can lead to localized flooding, stream channel destabilization, erosion, pollution, siltation, and sedimentation. All of these negative effects on the environment can and must be avoided by the careful design of an effective stormwater management system.

An efficient stormwater management plan considers both the quantity and quality of runoff collected. An efficient and cost-effective stormwater management program may entail a variety of design elements depending on site conditions, land use, and local regulations. Many of the components used in stormwater management systems can be manufactured using corrugated steel pipe (CSP). Stormwater management techniques can include structural design elements such as detention, retention, flow control, inlet protection, routing considerations, and water quality devices. Using CSP structures for design elements is an effective technique for achieving stormwater quantity and quality control.

Typical CSP underground detention system.

This chapter, while not intended to be a comprehensive evaluation of stormwater management practices, will examine various effective components using CSP that can be incorporated into a stormwater management program to satisfy most site development stormwater management needs.

CSP underground detention.

Urban stormwater runoff can commonly contain some of the following pollutants:

Bacteria: Bacteria are generated from animal and human waste, and can cause the degradation of receiving water bodies to the point that beaches and shellfish beds must be closed and increased costs may be incurred in the treatment of drinking water.

Chlorides: Salts that are applied to roads and parking lots in the winter appear in stormwater runoff in levels that can be higher than tolerable by many freshwater organisms.

Hydrocarbons: Oil and grease that leak from motor vehicles contain a wide variety of hydrocarbons that may be toxic to natural organisms.

Metals: Cadmium, copper, zinc and lead can often be found in the stormwater runoff from urban areas. All of these metals can be toxic to aquatic life when found in high enough concentrations.

Nutrients: Urban runoff has higher than normal levels of phosphorous and nitrogen, which in excessive levels can lead to increased algal growth in the receiving water bodies. Excessive algal growth can block sunlight from reaching underwater grasses and cause the depletion of oxygen. This process is called eutrophication.

Organic Carbon: Organic material, such as leaves and grass clippings get washed off of impervious surfaces during a storm. When these materials reach slow moving water bodies they settle out and decompose. The decomposition process depletes oxygen from the water. Water that has been depleted of oxygen can not support aquatic life.

Pesticides: Pesticides that are commonly used for landscaping and agricultural uses may be incorrectly applied and then washed off by stormwater. Pesticide laden stormwater is toxic to aquatic life.

Suspended Solids: Stormwater falling on impervious surfaces washes off sediment. Other pollutants will often bind to these sediments which will then be deposited into receiving water bodies. Suspended solids will remain in solution depleting water quality and other sediments will sink to the bottom diminishing the depth available for aquatic life.

Thermal Impacts: Because paved areas retain heat in higher quantities than natural groundcovers, the runoff from developed areas is typically higher than pre-development levels. Introducing higher temperature water to receiving water bodies can adversely affect cold water fisheries.

Trash and Debris: Trash and debris, while not necessarily toxic to living organisms can detract from the natural beauty of the environment and have significant impacts to socio-economic well being and tourism.

While it is not the intent of this chapter to present solutions that address all of these pollutants, the stormwater management systems discussed herein when used alone or in combination may help to prevent or ameliorate the effects of some of the pollutants.

The proper design of stormwater management systems must be undertaken to address both the quantities of pre-development, versus post-development flows and the quality of the post development discharges.

TERMINOLOGY

The following definitions apply throughout this chapter:

Best Management Practice (BMP): A Best Management Practice is a system of controls that seeks to prevent or mitigate water pollution from stormwater runoff.

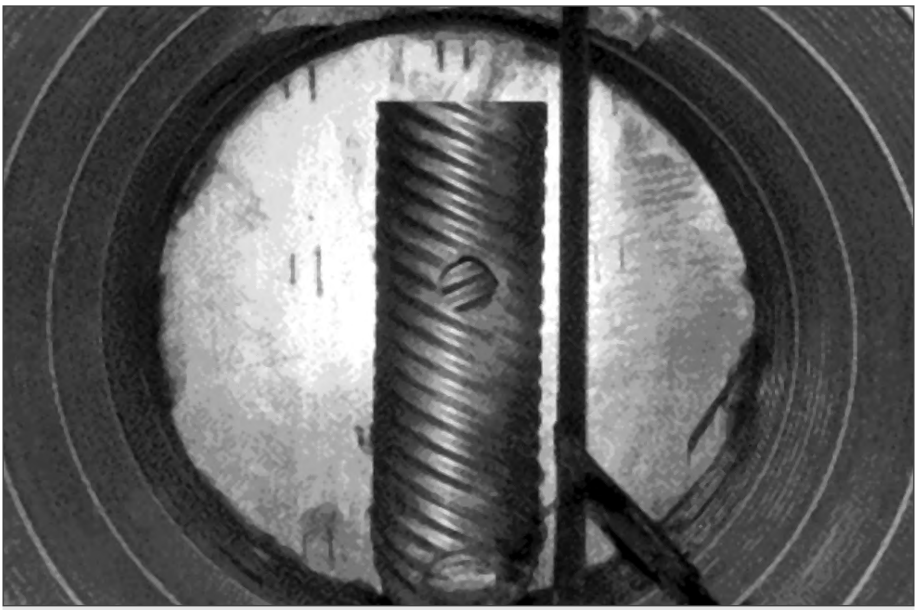
Non-point source discharge: Non-point discharge develops as rainwater, snowmelt or irrigation flows over land surfaces and through the ground, accumulating pollutants and then depositing those pollutants into receiving water bodies. Non-point sources deliver pollutants to surface waters from diffuse sources, including atmospheric deposition.

Total Maximum Daily Load (TMDL): The term TMDL refers to the calculated maximum amount of pollutants a water body may receive to attain or maintain its designated use.

Pre-development discharge: The pre-development discharge rate is the rate of stormwater flow from a project site prior to development activities that may be calculated using acceptable engineering practices. The discharge rate is affected by the return frequency of a particular storm event and its associated rainfall intensity, the cover of the land surface and its slope.

Post-development discharge: Quite often when land is developed there is an increase in the amount of impervious surface cover in the post-development state. That increased impervious area will alter the total amount of stormwater runoff from the development area. In most regulatory jurisdictions there are requirements that the rate of post-development runoff leaving a project site shall not exceed the pre-development discharge rate.

Detention System: A detention system provides a means of storing or detaining stormwater for a controlled release. Detention systems may consist of on-site ponds or below ground structures. CSP for below ground structures provides an economical, structurally sound method of collecting, storing and discharging stormwater that allows for greater utilization of land over an above ground storage system. Frequently, these systems are used under parking lots and recreation areas to maximize land usage. The purpose of a detention system is to manage the difference in the flow rates between the post-development and pre-development conditions. A detention system is designed to provide a proper volume of storage to attenuate the difference between a calculated peak post-development flow and an allowed pre-development flow rate. By providing the proper flow controls to limit the outflow from a detention system and the necessary volume of storage, the post-development flow rate may be managed to not exceed the pre-development rate.



■ Detention outlet control structure.

Typical CSP underground detention system being installed.

Retention System: A retention system consists of either (1) a sump graded into the landscape of a development site or (2) a below ground infiltration structure. The purpose of a retention system is to retain on the project site any increase in post-development versus pre-development stormwater runoff and allow the volume of retained stormwater to infiltrate into the surrounding soils. This allows for the recharging of underground aquifers and prevents downstream flooding from runoff on the post-developed site. Retention systems may only be used where there is adequate soil porosity to allow for the infiltration of the stormwater into the soil in a reasonable period of time. CSP provides an excellent method of achieving underground retention while allowing for utilization of the land above the system.

Water Quality devices: A variety of structures designed to remove pollutants through hydrodynamic forces or filtration. Water quality devices have differing capabilities for removing pollutants. They are useful as pretreatment and polishing systems, depending on their removal efficiencies and local regulations. Pre-treatment systems generally remove sediment, trash, debris and other suspended materials prior to detention or retention systems. Post-treatment or polishing systems are used to remove suspended materials as well as dissolved metals, nutrients, bacteria and chemicals. Post-treatment systems may be used as a stand alone system or in conjunction with detention or retention systems. CSP structures used for water quality include oil/sediment separators and sand filters. Oil/sediment separators are hydrodynamic structures which have been shown to be effective in sediment control and removal of floatables such as trash, debris and oils. Properly designed sand filters have been shown to effectively remove sediment and floatables, similar to the effectiveness of oil/sediment separators. They can also remove suspended and dissolved materials such as dissolved metals, nitrates, phosphates and organic material.

CSP sand filter system

HISTORY AND BACKGROUND

In 1972 amendments to the Federal Water Pollution Control Act, also know as the Clean Water Act, prohibited any discharges of pollutants to waters of the United States from a point source, unless the discharge was covered by a National Pollutant Discharge Elimination System (NPDES) permit. Initial efforts to curb pollution through the Clean Water Act concentrated on improving the quality of discharges from municipal sewage treatment plants and some industrial activities.

With the growing awareness that point sources were not the only significant source of pollutants, the Clean Water Act was amended in 1987 to address the impacts from stormwater runoff.

Phase I of NPDES

In 1990 the U.S. Environmental Protection Agency (EPA) developed Phase I of the NPDES Stormwater Program. The Phase I program addressed sources of stormwater runoff that had the greatest potential to negatively impact water quality. Under Phase I, the EPA required NPDES permit coverage for stormwater discharges from:

- Medium and large municipal separate storm sewer systems (MS4s) located in

incorporated places or counties with populations of 100,000 or more
- Eleven categories of industrial activities which includes construction activities that disturb five acres or more of land.

Phase II of NPDES

The Phase II Final Rule, published in the Federal Register on December 8, 1999, and enacted on March 10, 2003, requires NPDES permit coverage for stormwater discharges from:
- Certain regulated small municipal separate stormwater systems, and
- Construction activities disturbing between one and five acres of land (i.e., small construction activities).

Additionally, EPA Phase II guidelines list six areas that are to be addressed:
1. Public Education and Outreach
2. Public Partnership/ Involvement
3. Illicit Detection and Elimination
4. Construction Site Runoff Control
5. Post-Construction Runoff Control
6. Pollution Prevention/ Good Housekeeping

It is the above referenced Item 5 that this chapter most appropriately addresses. The ability to control post-construction runoff is dependent on the effective use of Best Management Practices (BMPs). These BMPs may be either structural or method based. The method based BMPs are outlines that may be used to control pollution discharges through the proper handling of materials and spill cleanup procedures. The structural BMPs may be proprietary, manufactured systems or they may be land-based that take advantage of ponds, swales and other water handling methods to produce a higher quality stormwater discharge.

Although most states are authorized to implement the NPDES stormwater program, there are several that have opted not to do so. In those states and territories that have opted to not implement the NPDES stormwater program, the EPA is the delegated authority.

HYDROLOGIC AND HYDRAULIC CONSIDERATIONS

The hydrologic and hydraulic design of construction sites using CSP is generally understood and has been covered in this manual in Chapters 3 and 5, respectively. This chapter will not reiterate those practices but will instead examine the various considerations necessary to evaluate and design water quality and quantity structures used for management of stormwater runoff. In general, the hydrologic considerations for stormwater management on construction sites require that the post-developed runoff be equal to or less than the pre-developed conditions (quantity control). This can be accomplished through a variety of designs including detention systems and retention systems. For water quality, regulations define a water quality volume to be treated as the first flush.

Hydrologic Considerations

Most municipalities and states have individual guidelines on the hydrologic parameters and design methods that must be used for water quantity and water quality. For water quantity control, the major consideration is determining the pre-development and post-development hydrograph.

For water quality designs, determining the water quality treatment volume is generally termed the first flush. The most common used definition is based on a volume over the watershed such as the first 1/2 inch of rainfall. Alternatively, the water quality treatment volume may be specified using the volume difference between the pre-developed and post developed run-off hydrographs for a specified rainfall intensity, duration, and return period. Regardless of the parameters and design methods, the calculations are considered for individual watersheds, sub-divided throughout the site.

Pre-treatment and Post-Treatment Hydraulics

The hydraulics affecting underground structures used in stormwater management systems generally fall into two categories: pre-treatment systems and post-treatment systems. Pre-treatment systems address water quality prior to storage or discharge from the site. These systems are concerned with only removing suspended pollutants from the water quality volume and may incorporate a bypass to accommodate large flows. Post-treatment or polishing systems can be any system designed to remove either suspended pollutants or dissolved pollutants. Filtration systems such as retention systems and sand filters manufactured using CSP are effective post-treatment methods.

COMPONENTS OF STORMWATER AND WATER QUALITY SYSTEMS

Detention/Retention Systems

As defined above, detention systems are systems that collect stormwater runoff from a site and control the release of the water at or below the pre-developed downstream discharge. Retention systems are systems that collect and hold stormwater until it can be released into the existing groundwater system. These systems do not have an outlet or limit the maximum discharge, so downstream flow patterns remain unchanged or reduced from the pre-developed flow patterns.

Typical CSP underground detention system.

Although there are a variety of products and methods available for detention and retention systems, piping systems offer several advantages. Land for development can be costly, and underground piping systems for retention or detention allow for more efficient use of land. Parking lots, landscaping, parks, and recreation areas can be utilized in the area over the storage system. Also, when comparing underground systems with above ground ponds, underground systems minimize the potential for insect breeding and eliminate the need for protective fencing. Underground piping systems are easily inspected and maintained to insure the long term performance of the hydrology and hydraulics of the site.

Recent developments provide detention facilities that are designed for optimal stormwater discharge rates. By quickly matching their actual discharge to the maximum regulated levels, these systems may reduce required storage capacity. They provide similar savings in below or above ground land use and construction costs. Underground detention systems not only reduce the space necessary for new facilities, they can be adopted to existing buried systems and ponds to expand capacities, allowing expansion of an existing development without adding storage capacity. These systems function automatically through flow controls. They do not require power, incur operator costs, or added maintenance costs.

Oil/Sediment Separator Structures

Oil/Sediment structures are useful as pre-treatment for the removal of sediment and floatables. They are basic treatment systems that remove sediment and debris in fluids using the principals of Stoke's law and Hazen's principal. Basically, Stoke's law analysis the rise (for floatable) or fall (for particles) in fluids based on their relative density to that of the fluid, the viscosity of the fluid and the shape of the floatable or particle. Therefore, using Stoke's law, the required velocity for settlement to occur can be calculated. Once the required settling velocity is determined, Hazen's principals can be applied to size an oil/water separator. Hazen's principal assumes that for a given flow and velocity, a separation area will be required to achieve the desired settlement. For stormwater applications, it is important to incorporate flow controls to insure the actual velocity through an oil/water separator is maintained to achieve the removal of floatables and particles.

To size oil/water separators, the design water quality flow, Qwq, should be provided, based on local or state regulations for first flush requirements. The only other data required is the targeted removal efficiency of the oil/water separator. In lieu of specific local regulations, since oil/water separators are generally considered pre-treatment devices, removal of a 50 sieve to a 200 sieve particle size is common. Higher removal rates may be designed but their actual efficiency is lessened, given the wide range of flows that occur in stormwater runoff.

Floatables such as oils, are captured in a separate chamber using a siphon or inverted weir. The primary function of these systems is to remove larger sediment and grit prior to entry into a stormwater storage system (retention or detention system) to enhance water quality and to prevent potential clogging of the storage system.

Sand Filters

Properly designed sand filters provide excellent filtration and pretreatment for stormwater quality. Sand filters remove debris and sediment in a separation chamber. Oils and floatables are removed using filtration through a sand medium, which can also be effective in removing some bacteria and dissolved chemicals. Therefore sand filters can be used for pretreatment or polishing systems. Sand filters are designed to treat a volume of water over a specific period of time. Various filtration media can be used in these systems. However, coarse grained sands are the most common because of their ability to maintain permeability and to effectively remove fine grained materials and some dissolved metals and chemicals. A typical sand filter design includes a pre-treatment sediment basin, a temporary ponding chamber, a filter bed with an underdrain and a clear well outlet. Most systems incorporate provisions for large flow by-pass and cleanouts for maintaining the system. There are three main types of sand filter designs: the Austin filter for large drainage areas (up to 20 acres), the Washington, DC sand filter, and the Delaware sand filter. The latter two are for smaller, urban drainage areas (1 acre or less). The Austin sand filter uses a sediment pond or an underground system with the sand filter as a polishing system. The Washington, DC and Delaware sand filters are similar in design and are a stand alone or combination water quality treatment system. Individual system designs are dependent on local regulations but the principals for design of sand filters are somewhat universal in application. In general, the concept is to calculate the volume of water to treat and the surface area required to treat that volume in a specified period of time. The time period should be based on a reasonable expectation of the interval between storm events and is usually over a 24 hour period. Longer periods may be used, as pollutants and sediment will not accumulate in this short an interval.

DESIGN OF COMPONENTS SYSTEMS

Detention/Retention Systems

Design of retention and detention systems is easily accomplished once the required data is obtained. The required data is the same for both retention and detention systems, but the designs vary slightly based on the outlet design.

Detention Design

Determine the required storage volume, V_{ds}, using the difference in the area of the post-developed versus the pre-developed hydrograph for a given design storm. An example of a typical pre-development and post-developed hydrograph is shown as Figure 6.1.

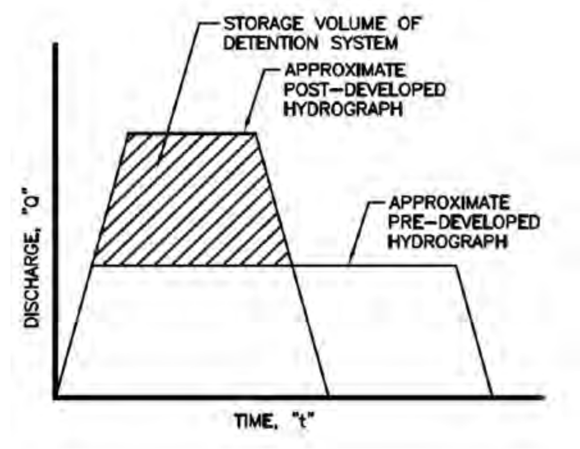


Figure 6.1 Post-developed vs. pre-developed hydrograph.

The pipe diameter, length, and number and extent of laterals can be determined from the following dimensional data:

- Inlet and outlet elevation
- Minimum cover required for pipe
- Length and width of area available for detention system

The design proceeds in the following steps:

(1.) Pipe Diameter. In general, the largest diameter and longest run of pipe will give the most economical design, as it will minimize the number of fabricated fittings for manifold headers. Select the largest diameter pipe as the difference in the inlet invert and the outlet invert. Make sure that elevations are such that minimum cover for the pipe can be provided.

$$D_{max} = \text{inlet invert} - \text{outlet invert}$$

(2.) Pipe Length. From the selected diameter, determine the total length of pipe required, L_{det}, using the storage capacity, SC, tabulated in Table 6.1.

$$L_{det} = V_{ds}/SC$$

(3.) Controlling Surface Dimension. From the area available for the detention system, select the controlling dimension, either length, L, or width, W.

(4.) Surface Area. The area required for the detention system is determined as the surface area per storage volume for the selected diameter. The surface area required per storage capacity is the ratio of center to center pipe spacing (s) per unit length divided by the storage capacity, SC, and can be calculated as:

$$A_{det} = V_{ds} * s/SC$$

(5.) Pipe Length and Width. Determine required length, L_{pipe}, or width, W_{pipe}, as follows:

For controlling length known, L_{pipe} is the available length less excavation overcut and width of header(s);

$$L_{pipe} = L - 2*d - n_{header}*OD$$

where: L = available length for detention
d = distance, trench sidewall to pipe (18 inches typical)
OD = average pipe outside diameter
n_{header} = number of manifold headers

For controlling width known, W_{pipe} is available width less excavation overcut;

$$W_{system} = W - 2*d$$

where: W= available width for detention
d = distance, trench sidewall to pipe (18 inches typical)

(6.) Laterals. Determine number of laterals (n) as:

$n = L_{det} / L_{pipe}$ (round down to nearest whole number > 1) for length control

or, $n = L_{det} / W_{system}$ (round down to nearest whole number > 1) for width control

(7.) Volume Check. Rounding down to the nearest whole number of laterals allows for the storage capacity in manifold headers. Detention systems use manifold headers to distribute stormwater between laterals. The number of manifolds used can be expressed as n_{header}. Therefore, the total storage volume provided, V_{det}, can be checked to insure the volume provided exceeds the required storage volume, V_{ds}, as:

$$V_{det} = \Sigma\ [\ (SC * L_{pipe} * n) + SC * (W_{ds} - 2*d) * n_{header}] \geq V_{ds}$$

Usually only one manifold header is required since the water level will equalize through the system. Using manifold headers on each end to increase flow routing through the system, however, is generally not economical due to increased fabrication costs and increased construction costs due to variations in manufacturing tolerances for pipe lengths.

Table 6.1

Pipe storage capacity (ft^3/foot of length)

Pipe Size	Pipe Spacing *	2 2/3 x 1/2	3 x 1	5 x 1	SRP 7 1/2 x 3/4
12	12	0.82	---	---	---
15	12	1.27	---	---	---
18	12	1.82	---	---	1.95
21	12	2.46	---	---	2.62
24	12	3.21	---	---	3.38
27	12	4.05	---	---	4.25
30	18	4.99	---	---	5.21
33	18	6.03	---	---	6.27
36	18	7.17	7.27	7.27	7.43
42	18	9.74	9.85	9.85	10.04
48	24	12.70	12.83	12.83	13.04
54	24	16.05	16.20	16.20	16.44
60	24	19.80	19.96	19.96	20.23
66	36	23.94	24.12	24.12	24.41
72	36	28.47	28.67	28.67	28.99
78	36	33.40	33.61	33.61	33.95
84	36	38.71	38.94	38.94	39.31
90	36	---	44.67	44.67	45.07
96	36	---	50.79	50.79	51.21
102	36	---	57.30	57.30	57.75
108	36	---	64.21	64.21	64.68
114	36	---	71.51	71.51	---
120	36	---	79.20	79.20	---
126	36	---	87.28	87.28	---
132	36	---	95.75	95.75	---
138	36	---	104.62	104.62	---
144	36	---	113.88	113.88	---

*Note: Typical pipe spacing may be reduced depending on the backfill material and compaction used for a specific project.

An example problem of a CSP detention system is included in the Section entitled Design Examples later in this chapter.

Retention Design

Retention system design is similar to detention system design except for the consideration of the additional storage area provided by the void area in the backfill and the permeability of the native soil. Selection of the required retention storage volume, V_{rs}, depends on whether an outlet replicating the pre-developed discharge is provided. If no outlet is provided, the required retention storage volume equals the total area under the post-developed hydrograph less the rate of percolation of the native soil, Figure 6.2. For the case where an outlet replicates the pre-discharge hydrograph, the required storage volume equals the difference between the post-developed and pre-developed hydrograph, Figure 6.3. The retention storage volume, V_{rs}, equals the pipe storage volume plus the storage volume in the voids of the backfill.

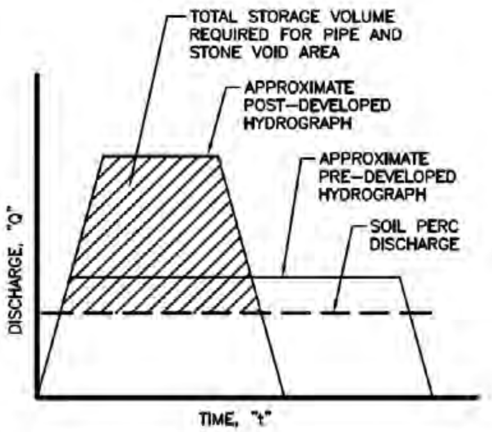


Figure 6.2 Post-developed vs. pre-developed hydrograph retention without outlet.

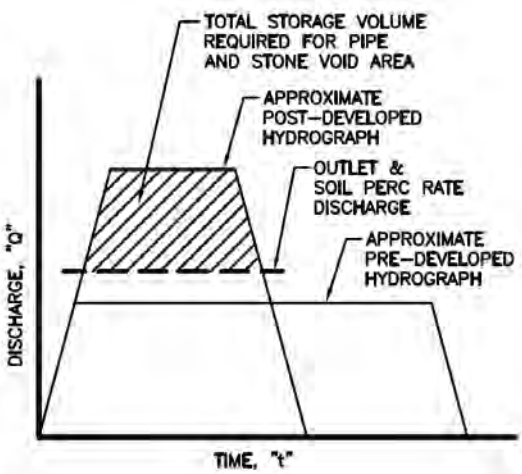


Figure 6.3 Post-developed vs. pre-developed hydrograph retention with outlet.

The pipe diameter, length, and number and extent of laterals can be determined from the following dimensional data:

- Inlet invert elevation
- Elevation of infiltration bed
- Minimum cover
- Length and width available for retention area
- Depth to water table

The design proceeds in the following steps:

(1.) Pipe Diameter. The largest diameter pipe will generally result in the greatest storage volume and the most economical installation. Select the diameter as the difference between the inlet invert less the elevation of the infiltration bed. Minimum cover should be checked for the diameter selected.

$$D_{max} = \text{inlet elev.} - \text{infiltration bed elev.}$$

(2.) Surface Area. From the selected diameter, determine the required surface area of the bed using the ratio of the center to center pipe spacing divided by the storage capacity of the pipe and backfill void space. The backfill void space varies depending on the backfill material used. Typical values for backfill porosities are listed in Table 2. Most aggregate producers can provide recommendations for porosity based on ASTM test methods. The surface (bed) area, A_{ret}, required for a retention system can be approximated as:

$$A_{ret} = V_{rs} * s/(SC + [(OD + s)*(OD + t) - SC]*n)$$

where: V_{rs} = volume required for retention
s = center to center pipe spacing
SC = storage capacity of pipe
OD = average pipe outside diameter
t = bedding thickness
n = porosity (void) ratio of backfill

The storage capacity of the pipe and backfill can be represented by the denominator in the equation above and is used in subsequent calculations. For convenience, the retention storage capacity, SC_{ret}, should be computed as follows:

$$SC_{ret} = SC + [(OD + s)*(OD + t) - SC]*n$$

(3.) Pipe Length and Width. Determine the total length of pipe required for the retention system, L_{ret},:

$$L_{ret} = V_{rs}/ SC_{ret}$$

For controlling length known, L_{pipe} is available length less excavation overcut and width of header(s)

$$L_{pipe} = L - 2*d - n_{header}*OD$$

where: L = available length for retention
d = distance, trench sidewall to pipe (18 inches typical)
OD = average pipe outside diameter
n_{header} = number of manifold headers

For controlling width known, W_{system} is available width less excavation overcut

$$W_{system} = W - 2*d$$

where: W= available width for retention
d = distance, trench sidewall to pipe (18 inches typical)

(4.) Laterals. Determine number of laterals (n) as:

For controlling length known

$n = L_{ret} / L_{pipe}$ (round down to nearest whole number > 1)

$W_{system} = L_{ret} / n$ (round down to nearest whole number increments that matches pipe spacing)

or, for controlling width known

$n = W_{system} / (OD + s)$ (round down to nearest whole number > 1)

$L_{pipe} = L_{ret} / n$ (round down to nearest whole number in 5 ft increments for optimum fabrication)

(5.) Volume Check. Rounding down to the nearest whole number of laterals allows for the storage capacity in manifold headers. Retention systems use manifold headers to distribute stormwater between laterals. The number of manifolds used can be expressed as n_{header}. Therefore, the total storage volume provided, V_{ret}, can be checked to insure the volume provided exceeds the required storage volume, V_{rs}, as:

$$V_{ret} = \Sigma \left[(SC_{ret} * L_{pipe} * n) + SC * (W_{ds} - 2*d) * n_{header} \right] \geq V_{rs}$$

An example problem of a CSP retention system is included in the Section entitled Design Examples later in this chapter.

Table 6.2

Typical porosity for various backfill materials*

Description	Porosity (n)
Uniform Graded Sand, Compacted	0.35
Well Graded Sand, Compacted	0.30
Uniform Graded Stone, Compacted	0.40
Well Graded Stone, Compacted	0.28

Note: Values are for typical backfill materials based on specific gradations for aggregates available from most quarries.

Oil/Sediment Structure Design

The design of oil/sediment structures is based on either the rise rate of floatables (oils) or the fall rate of solids (sediment), depending on the targeted removal criteria. Regardless, the calculation for either the rise rate or fall rate is accomplished using Stoke's Law. The required velocity for the rise rate or fall rate is determined based on particle size and its relative density to the stormwater as:

$$V_{(rise/fall)} = [g(\rho_w - \rho_{r/f})d^2/18\mu]$$

where:
$V_{(rise/fall)}$ = rise or fall rate of targeted removal material (ft/sec)
g = acceleration due to gravity (ft/sec^2)
ρ_w = density of water (slug/ft^3)
$\rho_{r/f}$ = density of targeted material (slug/ft^3)
d = diameter of targeted material (ft)
μ = viscosity of water at 68°F

Once the required velocity for settlement is determined, the horizontal settlement area required to achieve separation can be calculated. The horizontal area as opposed to cross-sectional area controls separation. An excellent derivation of the theory of separation using Stoke's Law and the horizontal area of an oil-water separator is presented in the American Petroleum Institute's Publication 421, Feb 1990, and is adapted here for use in circular pipe oil-water separators.

Separation occurs when residence time, t_r, exceeds the residence time, t_s.

$$t_s \leq t_r$$

Theoretically, a particle only has to fall below the elevation of the weir to settle, since at that point the particle would impact the weir and terminate its horizontal velocity and fall. However, to account for velocity acceleration over the weir, it is advisable to assume settlement would not occur until the particle falls farther below the weir. For oil-water separators, the weir is set a minimum of 1 foot below the top of the pipe or at the invert of the inlet pipe, whichever is greater. Therefore, it is conservative to assume settlement

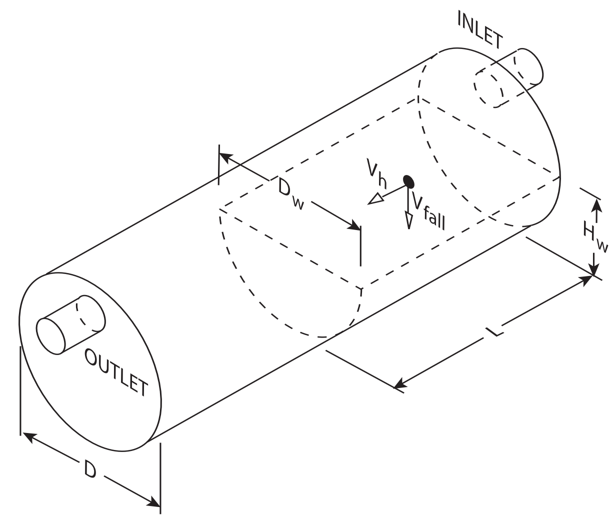


Figure 6.4 Schematic for oil-water separator.

will occur when the particle drops to one-half the diameter. The separation time, t_s, and residence time, t_r, can be calculated from the particle velocities, V_{fall} and V_H, and separation length, L (Figure 6.4), as:

$$t_s = d/2^*V_{fall}$$

$$t_r = L/V_H$$

Therefore, to insure separation

$$d/2^*V_{fall} \leq L/V_H$$

Solving for the settling velocity

$$V_{fall} \geq V_H\ (d/2^*L)$$

Now for continuity, the horizontal velocity (assuming uniform flow) is the flow divided by the cross-sectional area:

$$V_H = Q_{wq}/\pi^*(d^2/4)$$

By substitution

$$V_{fall} \geq Q_{wq}\ ^*d/(2^*L^*\pi^*d^2/4) = 2Q_{wq}/(L^*\pi^*d)$$

The horizontal area, A_H, at the center of the pipe is:

$$A_H = L*d$$

This assumes uniform flow through the pipe. However, API recommends a dimensionless factor of 1.2 to 1.75 be applied. Since the relative length to diameter is usually large, and to make the factor of $2/\pi$ uniform, a 1.57 factor is recommended. Then, the velocity necessary for settlement becomes:

$$V_{fall} \geq Q_{wq} /(L*d) = Q_{wq} /A_H$$

Now, the velocity calculated by Stoke's Law can be determined to be less than V_{fall}:

$$V_{(rise/fall)} = [g(\rho_w - \rho_{r/f})d^2/18\mu] \leq Q_{wq} /(L*d) = Q_{wq} /A_H$$

Once the required velocity through the oil/sediment is known, the size of the oil/sediment structure is based on the plan view area of the oil/sediment chamber. The time the targeted particle resides in the oil/sediment chamber must be less than the time required for the rise or fall to occur.

An example problem of a CSP oil-water separator system is included in the section entitled Design Examples later in this chapter.

Sand Filter Design

The design for sand filters will focus on small stand alone treatment systems using the Washington, DC and Delaware sand filter systems. This design can easily be adapted to larger systems used in combination with detention systems for storage and pretreatment. The basic information needed for design is the water quality volume, W_{QV}, and the duration of the discharge through the filter media. This chapter will not attempt to select a W_{QV} as that is usually dictated by the local regulatory agency. However, based on typical applications, the volumetric approach most commonly used is to select a rainfall volume for the percent impervious of the drainage area, which will allow calculating the volume on the watershed to be treated. This would be the first flush in a volumetric approach which has been discussed in earlier sections of this chapter. Once the volume to be treated is calculated, selection and design of the sand filter system is relatively straightforward by varying proportional relationships between surface area and permeability.

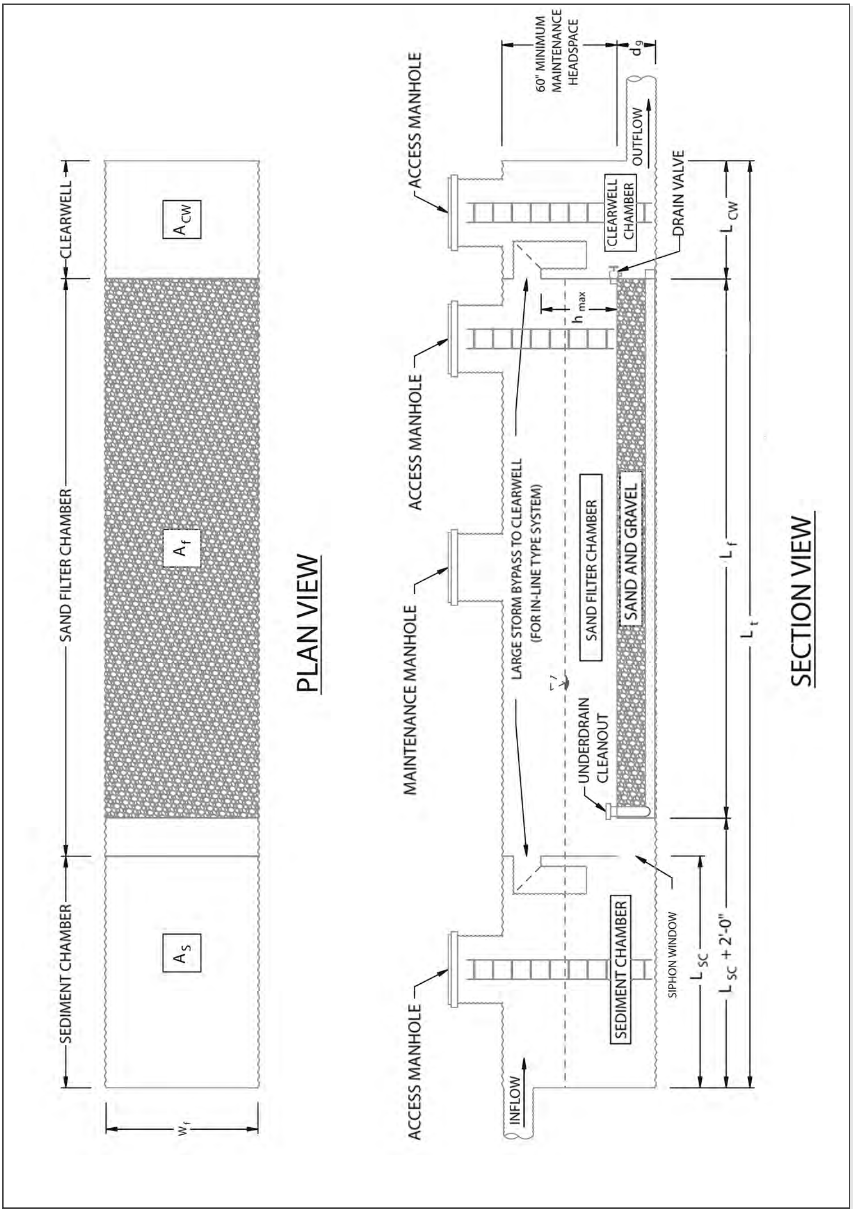
Design calculations may proceed as follows.

(1.) Determine minimum surface area of sand filter:

$$A_f = W_{QV}*d_{fg}/[k*(h_f + d_{fg})*t_f]$$

where: W_{QV} = water quality volume

d_{fg} = depth of filter media (sand & gravel)
k = permeability of filter media, typically between 2 to 4 ft/day (3.5 ft/day normally used for design)
h_f = average height of water above sand filter (distance from top of filter to overflow)/2
t_f = filtration time (user defined, typically 40 hrs)



■ **Figure 6.5** Typical sand filter.

(2.) Determine width of sand filter:

$$W_f = 2[2 * r * d_{fg} - d_{fg}^2]^{1/2}$$

where: r = radius of pipe
d_{fg} = depth of filter media (sand & gravel)

(3.) Determine length of sand filter:

$$L_f = A_f/W_f$$

(4.) Determine length of sediment chamber:

Since sand filters are designed for long filtration times (t_f), sediment chambers are sized as a percentage of the water quality volume, typically 20% of the water quality volume.

An example problem of a CSP sand filter system is included in the section entitled Design Examples later in this chapter.

INSPECTION, OPERATION AND MAINTENANCE

Stormwater Management Systems

Most stormwater management systems are designed to be self-operating without the need for maintenance personnel to be involved in day-to-day operations. This is especially beneficial because of the sporadic nature of rainfall occurrences. It is undesirable to have a stormwater management system that requires human intervention when weather conditions might be the most adverse. Although there are some emergency flow control devices that require manual operation, the most effective designs are those that automate even the emergency devices. An example of this is the use of an emergency stop valve at a bulk petroleum distribution facility. Normal operations at one of these facilities may allow stormwater runoff to flow unrestricted from the site to a receiving water body. To prevent a catastrophic spill at this type of facility there may be a valve that could be manually closed to allow spilled materials to be contained within a secondary containment system. What would be preferable to the manual valve would be a commercially available automatic valve that would sense the difference in the specific gravities between ordinary stormwater runoff and the lighter petroleum products and automatically close, thus allowing for the retention of the spilled material on site.

The operation of other stormwater management systems may be as simple as ensuring that drainage piping and culverts remain free of accumulated debris, to as complex as the rehabilitation of failing culverts. To ensure the proper operation of storm drains and culverts, visual inspections of all facilities within a given jurisdiction should be undertaken on a yearly basis. Reports of flooding upstream of structures should be compiled and tracked and used for a regular analysis of the particular system.

If frequent flooding is reported upstream of a particular structure, then consideration should be given to a more extensive inspection, which may require the use of video cameras. Video cameras may be introduced to a culvert or drain line and can provide a record of the interior of the structure with corresponding stationing that can be used to identify problem areas.

Maintenance of culverts quite often requires the use of high pressure water jets and vacuum trucks to free and remove accumulated debris. High pressure water jets often employ the use of jetting heads that will direct a spray backward towards the starting point of the operation and propel the head forward into the culvert. As the jetting head moves forward the water that is directed backward moves debris toward the starting point where it may be vacuumed out. In some cases a rotary root cutter may be used to remove roots that may find their way into the cracks between pipe sections.

Failing culverts are often replaced by a full depth excavation and replacement. Sometimes it is more cost-effective to repair the culvert with a relining operation. In this case a pipe of somewhat smaller diameter is inserted into the culvert, sometime employing the use of guide rails to maintain the correct position within the culvert. When the relining pipe is completed inserted into the culvert, a grout mix is pumped through plugs in the structure to create a self contained replacement culvert. In this manner, the strength of the replacement pipe may be relied upon to carry the full load.

Water Quality Systems

The maintenance of water quality systems depends upon the nature of their construction. Land based systems that employ wet ponds, swales or detention ponds will require excavation of accumulated debris, the replacement of soils, and replanting of native vegetation or wetland plants. Manufactured systems are typically more easily maintained through the use of a vacuum truck and high pressure water.

In the case of either land based, or manufactured systems, detailed inspection and maintenance logs should be kept to allow for the review of successfulness of the devices and to assist in the scheduling of future maintenance events.

DESIGN EXAMPLES

Detention System Design Example Using Corrugated Steel Pipe

Given: Inlet elevation = 100.0 ft
Outlet elevation = 95.3 ft
Detention Volume Required Vds = 3,500 ft^3
Available length for detention system = 45ft
See section on Detention Design for equations used below.

Determine maximum pipe diameter:

D_{max} = 100.00 – 95.3 = 4.7 ft (note system size could be increased by size of inlet pipe)

Try 54 inch diameter 3x1 inch corrugated steel pipe
From Table 6.1, Storage Capacity, SC = 16.2 ft^3/ft

Calculate total pipe length required, L_{det}:

$$L_{det} = 3{,}500/16.2 = 216 \text{ ft}$$

From the available length for the detention system, calculate actual length of detention system with manifold header on each end:

L_{pipe} = 45 – 2 (1.5) – 2 (56/12) = 32.7 ft   Use 32 ft laterals.

Determine number of laterals, n:

n = 216/32 = 6.75   Use 6 laterals.

Determine width of system header:

$$W_{sys} = 6\ (56/12) + 5\ (2.25) = 39.25 \text{ ft header}$$

Calculate total storage volume provided:

$$V_{det} = [32\ (16.2)\ (6)] + [(39.25)\ (16.2)\ (2)] = 4382 \text{ ft}^3 > 3{,}500$$

Since there were two headers, the volume provided is greater than required and it may be possible to eliminate a lateral to provide a more economical system. Therefore, recalculate with 5 laterals.

$$W_{sys} = 5\ (56/12) + 4\ (2.25) = 32.33 \text{ ft header}$$

And

$$V_{det} = [32\ (16.2)\ (5)] + [(32.33)\ (16.2)\ (2)] = 3639 \text{ ft}^3 > 3{,}500 \text{ required}$$

Therefore; use 54 inch diameter 3x1 inch corrugated steel pipe with five 32 foot laterals and two manifold headers as shown in Figure 6.6.

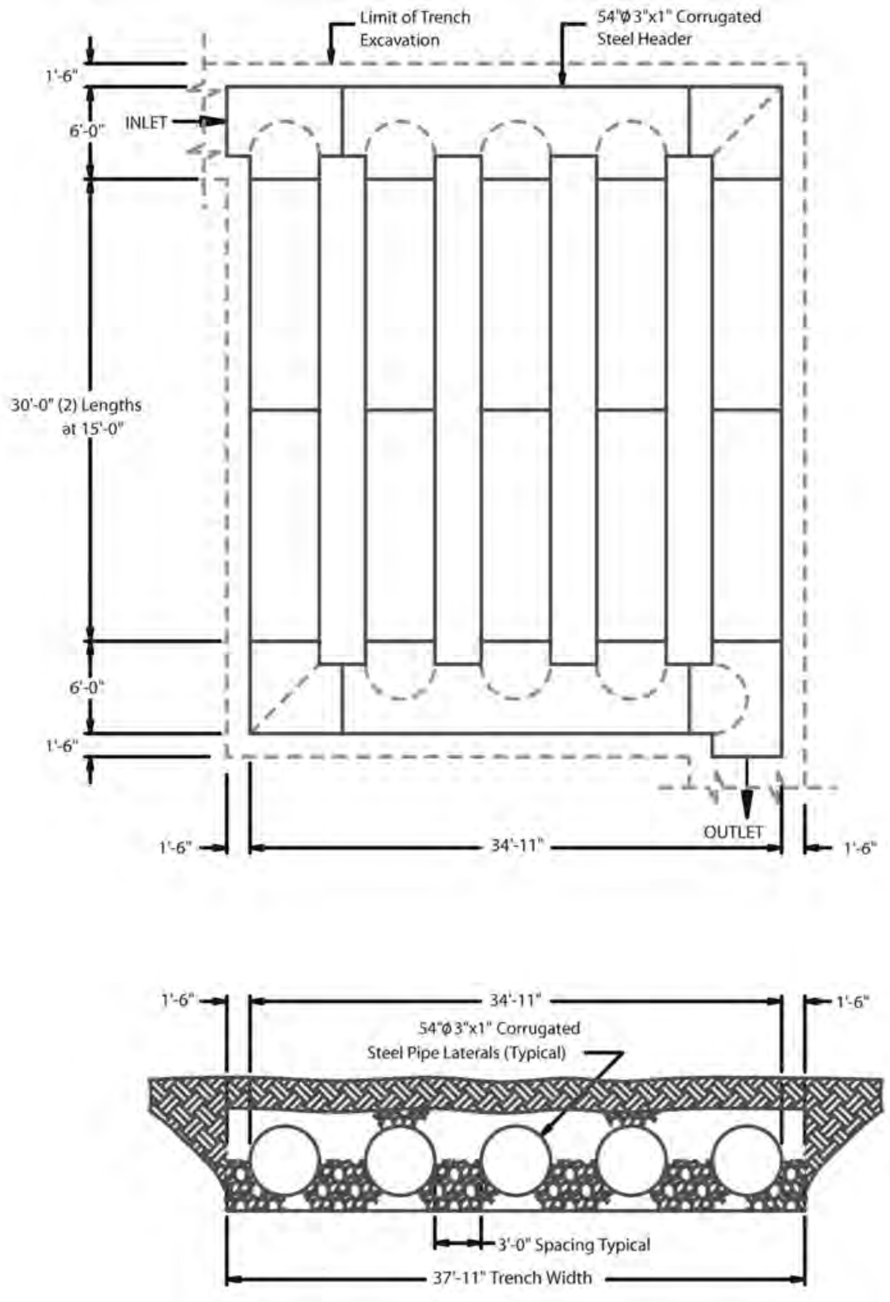


Figure 6.6 Detention system layout for design example.

Retention System Design Example Using Corrugated Steel Pipe

Given: Inlet elevation = 60.0 ft
Infiltration bed elevation = 52.0 ft
Retention Volume Required V_{rs} = 150,000 ft^3
Available width for retention system = 100 ft
See section on Retention Design for equations used below.

Determine maximum pipe diameter:

D_{max} = 60.00 – 52.0 = 8.0 ft (note system size could be increased by size of inlet pipe)

Try 96 inch diameter 5x1 inch perforated corrugated steel pipe with 1 1/2 inch clean washed stone having a void ratio of 40%. From Table 6.1, Storage Capacity, SC = 50.79 cf.

Calculate Area of retention bed, A_{ret}:

$$A_{ret} = [150{,}000\ (98 + 36)/12]/[50.79 + [(98 + 36)\ (98 + 6)/144 - 50.79]\ (0.40)] = 24{,}210\ ft^2$$

Determine total length of perforated pipe required for retention system:

$$L_{ret} = 150{,}000/[50.79 + [(98 + 36)\ (98 + 6)/144 - 50.79]\ (0.40)] = 2168\ ft$$

Since the width is limited to 100 ft, determine actual retention system width, W_{sys}:

$$W_{sys} = 100 - 2\ (1.5) = 97\ ft$$

Determine number of laterals required, n:

$n = 97/[(98+36)/12] = 8.7$ Use 8 laterals with one header.

Determine length of laterals, L_{pipe}:

$L_{pipe} = 2168/8 = 271$ ft Try 265 ft. laterals.

Determine retention volume provided, V_{ret}:

$$V_{ret} = 265\ (8)\ [50.79 + [(98 + 36)\ (98 + 6)/144 - 50.79]\ (0.40)] + 50.79\ [(98)(8)/12 + (36)\ (7)/12] = 151{,}057\ cfs > 150{,}000\ ft^3\ \text{OK}$$

Use 8 laterals of 96 inch diameter 5x1 inch perforated corrugated steel pipe at 265 feet long with 1 1/2 inch clean washed stone backfill. To insure adequacy of the design, the infiltration rate of a 97 foot by 265 foot bed should be verified.

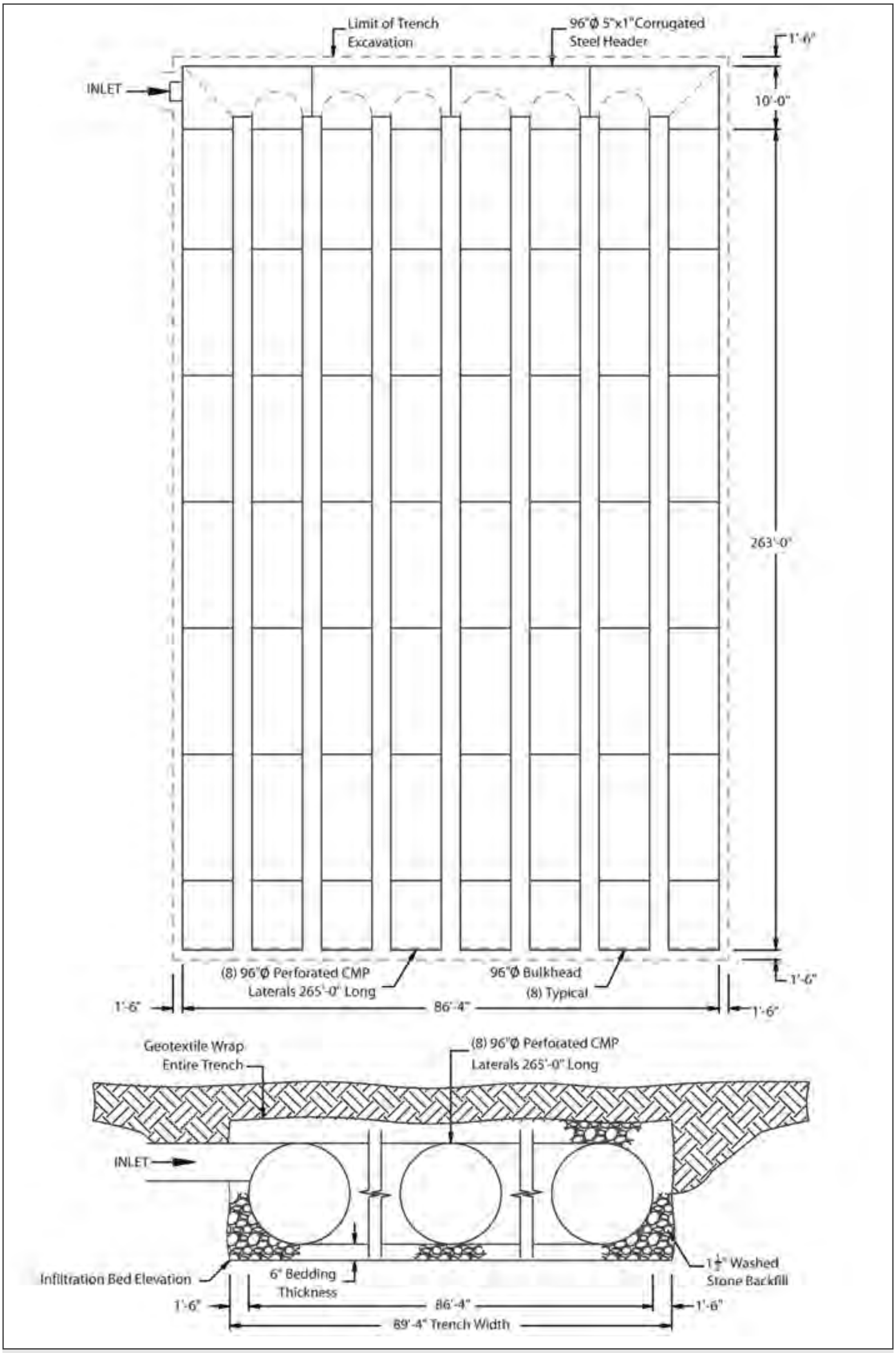


Figure 6.7 Retention system layout for design example.

Design of CSP Oil/Sediment Separator Structure

Given: Targeted particle size = 140 sieve = 0.000342 ft
Water quality flow, Qwq = 2.3 ft^3/s
Density – soil, rsoil = 120 lb/ft3
Density – water, rwater = 62.4 lb/ft3
Viscosity of water @ 68o F, m = 2.1 x 10-5 lbf-sec/ft^2
See section on Oil/Sediment Separator Structure Design for discussion of equations used below.

Convert density to slug/ft^3, 1 slug = 32.17405 ft/sec^2:

Density – soil, ρ_{soil} = 120/32.17405 = 3.73 slug/ft^3

Density – water, ρ_{water} = 62.4/32.17405 = 1.94 slug/ft^3

Determine required settling velocity:

$$V_{(rise/fall)} = g(\rho_{soil} - \rho_{water})d^2/18\,\mu$$
$$= 32.2(3.73\text{-}1.94)(0.000342)^2/18(2.1\text{x}10^{-5})$$
$$= 0.0178 \text{ ft/sec}$$

Determine horizontal settlement area required:

$V_{(rise/fall)} > Q_{wq}/(A_H)$
Assumes particle falls 1/2 of pipe diameter and turbulence factor of 1.57.

$0.0178 > 2.3/(A_H)$

Solving for A_H

$A_H > 2.3/0.0178 = 129.2 \text{ ft}^2$

Try 48" diameter pipe and find length :

L = 129.2/4 = 32.3 ft

Note: If turbulence is eliminated and fall is limited to the top of the weir (1 ft) L = 32.3/1.57(2/1) = 10.29 ft. However, this is not as conservative. Additionally, in lieu of sizing the oil chamber, it is common practice to make them 1/3 of the sediment chamber since the predominant runoff for most sites is sediment.

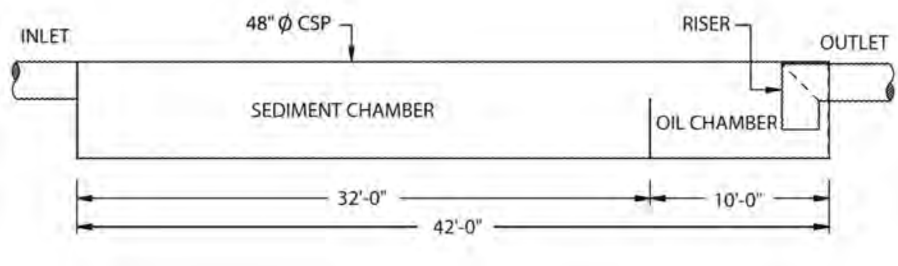


Figure 6.8 Schematic of Oil/Water Separator

Design of CSP Sand Filter

Given: Assume a 1 acre site that is 80% impervious and a 1 inch rainfall. The inlet is an 18 inch diameter pipe with the invert 2 feet below grade. The maximum distance to the outfall is 8 feet below the invert of the inlet pipe. Try a 96 inch diameter sand filter.

d_{fg} = depth of filter (typically 1.5 to 2.0 ft) = 2.0 ft
k = permeability of filter media = 3.5
(typically 2 to 4 ft/day, 3.5 ft/day normally used for design)
t_f = filtration time (user defined, typically 24 to 48 hours) = 48 hours
See section on Sand Filter Design for equations used below.

Determine water quality volume:

$$W_{QV} = (\text{\% impervious}/100)(\text{rainfall})(\text{site acres})(43{,}560\ \text{ft}^2/\text{acre})$$
$$= (80/100)(1/12)(1)(43{,}560)$$
$$= 2{,}904\ \text{ft}^3$$

Determine height to bypass, h_{max}:

$$h_{max} = D - d_{fg} - \text{dinlet}$$
$$h_{max} = (96/12) - 2 - 1.5 = 4.5\ \text{ft}$$

Average height over filter, h_f:

$$h_f = h_{max}/2$$
$$h_f = 4.5/2 = 2.25\ \text{ft}$$

Determine minimum surface area:

$$A_f = W_{QV} d_{fg}/[k(h_f + d_{fg})t_f]$$
$$= 2{,}904\ (2)/[(3.5)(2.25{+}2)(48/24)] = 195.2\ ft^2$$

Determine width of filtration bed:

$$W = 2(2rd_{fg} - d_{fg}{}^2)^{1/2}$$
$$= 2[2(4)(2.0) - (2.0)^2]^{1/2}$$
$$= 6.93\ ft$$

Determine length of filtration bed:

$$L = 195.2/6.93 = 28.2\ ft \quad \text{Say } 28.5\ ft$$

Determine sediment chamber length as 20% of W_{QV}:

Volume required = 2,904(0.20) = 581 ft^3

Length of chord at h_{max} elevation
c = 2 [(8/2) – 2.52]1/2 = 6.24 ft

Area in sediment chamber = $1/2\ \pi\ (8/2)^2 + 1/2\ (6.48 + 8)(8/2 - 1.5) = 43.2\ ft^2$

Length of sediment chamber = 581/43.2 = 13.4 ft. Say 13.5 ft

Total length of sand filter structure including a 5 foot length of clearwell for access:

L = 28.5 + 2 + 13.5 + 5 = 49.0

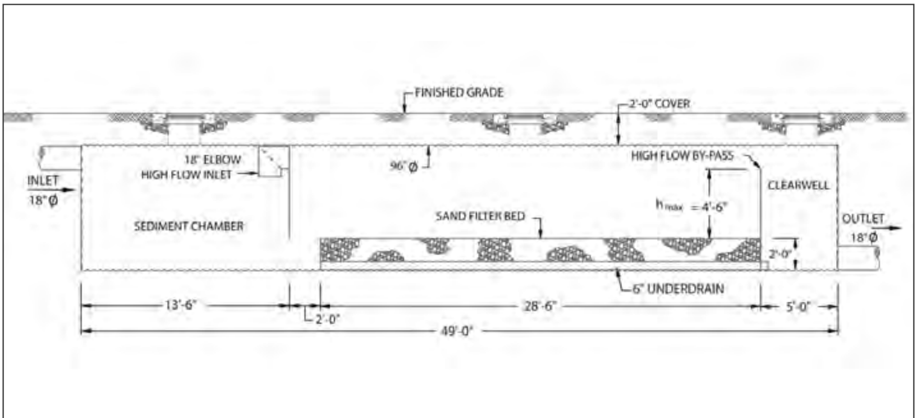


Figure 6.9 Sand filter layout for design example.

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American Petroleum Institute, "Design and Operation of Oil-Water Separators," Publication 421, Feb. 1990.

US Environmental Protection Agency, "Stormwater Technology Fact Sheet – Sand Filters", Sept. 1999.

Typical CSP underground detention system for a new housing development.

■ **Figure 7.1** A corrugated steel long span structure will soon be a stream crossing under a freeway.

CHAPTER seven

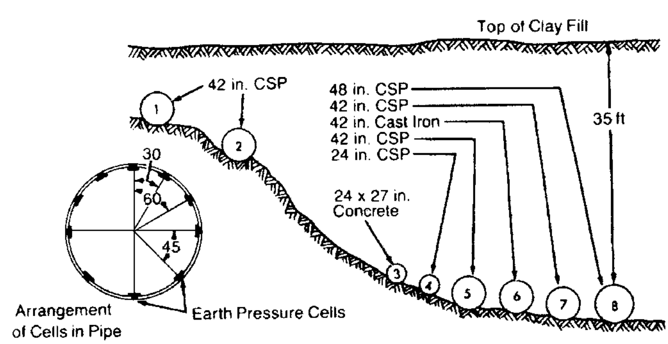
Structural Design

INTRODUCTION

Corrugated steel pipe has long been recognized for superior strength to withstand both high live loads and deep burial soil loads. Research over the last several decades has shown that this strength is the result of a complex composite behavior – the interaction of soil with the steel structure. In spite of this complex behavior, simple conservative design methods have been developed and are in widespread use. These are the methods generally adopted by specifying agencies such as the American Association of State Highway and Transportation Officials (AASHTO) and the American Society for Testing and Materials (ASTM), as discussed further in this chapter. However, CSP products can also be designed and evaluated by the newer evolving methods, such as Finite Element Analysis (FEA). The CANDE computer program and the SCI (Soil Culvert Interaction) design method represent applications of FEA to CSP design. A design method developed by the American Iron and Steel Institute (AISI), which is described in detail herein, has been used successfully for standard pipe, arches, and pipe arches for many years.

It has been well established that the main function of the soil is to provide lateral support to the steel pipe, maintaining its shape so the pipe wall acts as a compression ring. Design checks are made to ensure that the pipe wall has the required resistance to crushing and instability. Bending moments are generally disregarded except for certain types of structures such as box culverts, deep corrugated structures, and some long span structures.

When corrugated steel pipe was introduced over 100 years ago, early strength tests were quite crude and included circus elephants balanced on unburied pipe and threshing rigs placed over shallow buried pipe. However, it wasn't long before laboratory hydraulic and sand box tests were performed by Talbot, Fowler and others, as well as evaluations under deep fills by Iowa State University (Marston, Spangler et al, 1913) and the University of North Carolina (Braune, Cain, Janda). Large scale field tests under the Illinois Central



■ **Figure 7.2** American Railway Engineering Association tests on culvert pipe at Farina, Illinois. Readings were taken on earth pressure cells.

Railroad (1923) demonstrated early on that corrugated steel pipe carries only a portion of the expected load. These tests indicated that the pipe typically carried about 60 percent of the load and the backfill envelope carried the other 40 percent.

As mentioned above, finite element analysis has been used in more recent times to investigate the behavior of corrugated steel pipe. With this method the pipe and the surrounding backfill are broken into discrete structural elements with known properties and a computerized matrix analysis is used to solve for the forces in each element. Thus, detailed information is obtained on forces, bending moments, and soil pressures. This is the basis of the computer program CANDE (Culvert Analysis and Design), initially developed by M. G. Katona et al for the Federal Highway Administration (FHWA) in 1976. Continuing interest in this approach led to improved versions, culminating with the development of a new user friendly version of CANDE released in 2008.

The SCI (Soil Culvert Interaction) method developed by J. M. Duncan et al in 1978 is also based on the finite element approach. In this case, Prof. Duncan and colleagues ran numerous cases on the computer and synthesized the results into a set of equations and charts to determine maximum force and bending moment. Although lengthy, this method can be used to obtain a hand solution. It is particularly useful for investigating minimum cover situations with high live loads. This work is the basis of the method specified for design in the Canadian Highway Bridge Design Code (CHBDC).

In spite of the strength derived from the backfill envelope, much of the design emphasis today still concentrates on selecting a steel structure with adequate strength to carry the loads and an adequate stiffness to allow it to be installed while maintaining its shape. While the backfill envelope is a substantial portion of the final strength, it need only be adequate to support the corrugated steel pipe, allowing it to function in ring compression. The design procedures found in the specifications of AASHTO and ASTM are based on this concept.

Three design procedures are available in AASHTO. The traditional Service Load Design (SLD) procedure, also known as Allowable Stress Design (ASD) and Load Factor Design (LFD) are both found in the AASHTO Standard Specifications for Highway Bridges. Load and Resistance Factor Design (LRFD) is found in the AASHTO LRFD Bridge Design Specifications. AASHTO's goal is to use LRFD design for all new construction. ASTM Standard Practice A796/A796M includes both ASD and LRFD as alternatives. In many cases, the result obtained by each of these procedures is similar when similar loadings are used.

SALIENT RESEARCH

In addition to the development of design methods noted previously, there have been many additional studies made on the performance of corrugated steel pipe. Three of these salient research studies are noted on the following pages.

Utah State Test Program

Extensive research was conducted at Utah State University by Dr. R. K. Watkins and associates during 1967 – 1970 under the sponsorship of the American Iron and Steel Institute. This was the first time that numerous full-size CSP installed in a backfill were loaded to their ultimate performance limit in a field laboratory. Approximately 130 pipes, 20 feet long, in sizes from 24 inch to 60 inch diameter were loaded to their performance limit in low grade soil backfills compacted from 70% to 99% standard AASHTO density. Riveted, spot welded and helical pipe fabrications were included in both 2-2/3 x 1/2 inch and 3 x 1 inch corrugations. Confined compression tests were made on six different soils to correlate results to commonly used backfill materials.

The pipes were installed and loaded in a 24 foot long, 15 foot wide and 18 foot high test cell constructed of 5/8 inch steel plate of elliptic cross-section. (Figure 7.3). Steel trusses pinned to the top of the cell walls supported hydraulic cylinders, which applied a uniform pressure up to 20,000 psf on the upper surface of the soil. The backfill material was a silty sand installed in lifts and compacted with manually operated mechanical compactors. Compactive effort and moisture contents were varied to obtain densities from 70% to 99% standard AASHTO.

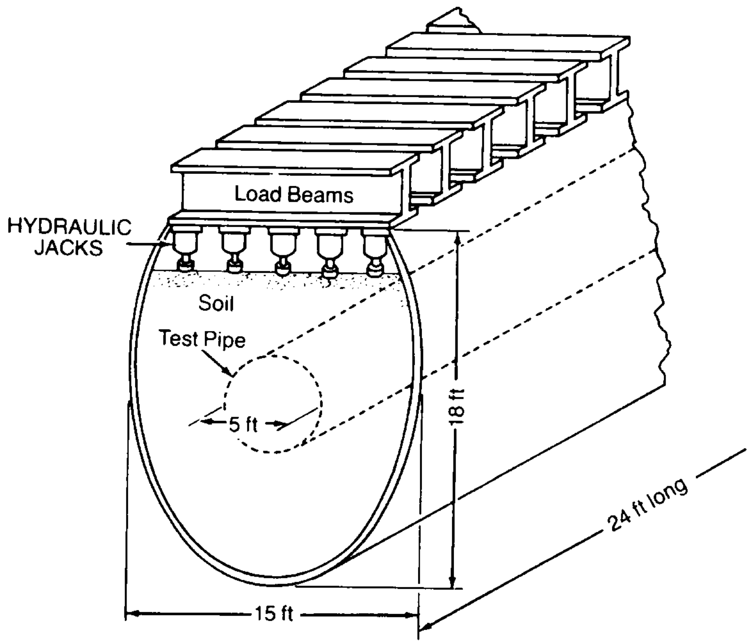


Figure 7.3 Diagramatic sketch of test cell showing method of applying load with hydraulic jacks. The cell was buttressed with reinforced concrete retaining walls and wing walls. Test were performed by Engineering Experiment Station of Utah State University for American Iron and Steel Institute.

After backfill, steel plates were placed on top of the soil to improve the bearing of the hydraulic rams. Load was applied in planned increments with the following readings taken: loading force, soil pressure on the pipe, vertical pipe deflection, and ring profile. Testing was terminated when the hydraulic ram pressure could no longer be increased. It is significant that, in this condition, the pipe could continue to deform in the test cell. Soil arching made the structure stable under applied loads much higher than those recorded in the test.

Results of the test plotted for five degrees of standard AASHTO density for the backfill are shown in Figure 7.4. Assuming the load applied by the hydraulic rams equals the pressure acting on the pipe, the ultimate steel stresses are plotted on a buckling chart. It is

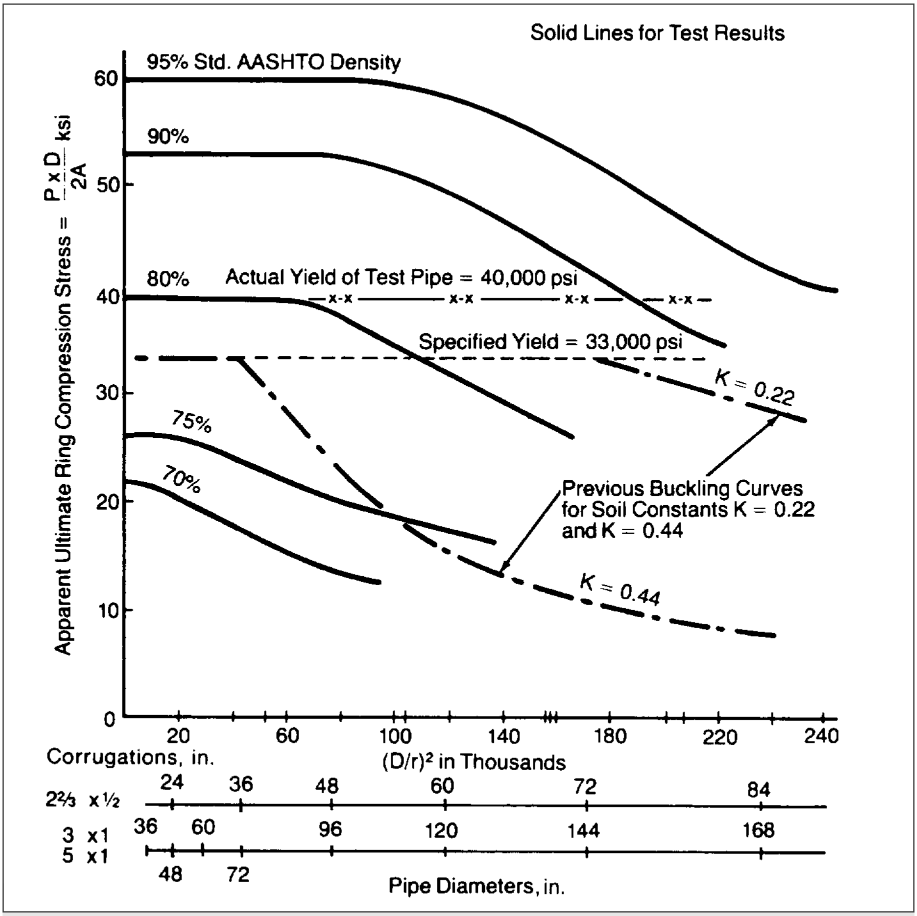


Figure 7.4 Results of Utah loading tests on corrugated steel pipe, showing apparent ultimate ring compression stress as a function of diameter and corrugations of various values of soil density determined by AASHTO standards.

immediately apparent that most of the steel stresses calculated by this criterion, are fictitious because they greatly exceed the yield point. This is explained by Figure 7.5, which illustrates how the applied load is actually carried in part by the soil arch formed in the compacted backfill as load is applied thereto and pipe and soil strains occur. Because the stresses on the ordinate in Figure 7.4 are calculated from the total load, with no reduction taken for the load carried by the soil in arching action, they are designated as apparent stresses.

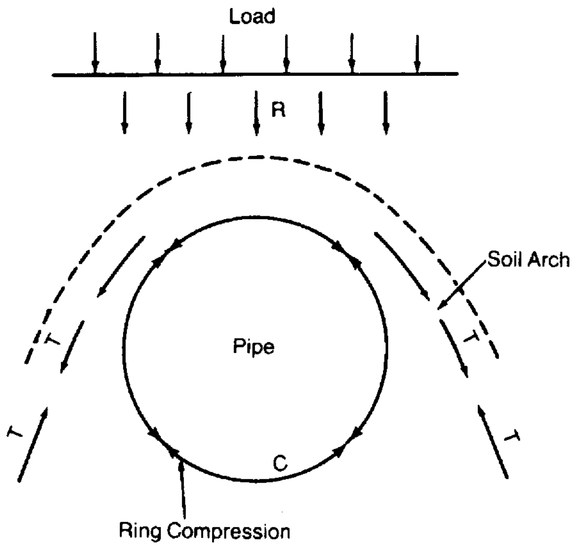


Figure 7.5 Diagram showing how load Pv is partly carried by means of a soil arch over the pipe.

A prime objective of the Utah program was to establish a practical correlation between backfill density and pipe behavior. The Utah program provided, for the first time, ultimate performance data on full scale soil-steel installations, utilizing a low-grade backfill soil and normal field methods and equipment. The Utah research confirmed what has been observed in field installations for decades. The quality and density of backfill required to permit the pipe to carry high stress levels, to or near the yield point, is of ordinary magnitude comparable to current common practices for most highway embankments. The test results (Figure 7.4) are plotted on an outdated buckling stress graph where dashed lines show buckling curves that were correlated to an unrealistically high level of soil compaction. The wide disparity between the K = 0.44 curve for 85% compaction and the actual performance results at 85% is readily apparent.

This research established a zone of "critical density" between 70% and 80% standard AASHTO density. The critical density represents a level of backfill compaction that will allow the pipe to carry ring compression stress at or near the yield point. At a conservative value of 80% standard AASHTO density, there is enough soil support to preclude deflection collapse and the pipe carries stress near the yield point.

The test soil used in the Utah research was considered a low grade material for pipe backfill. Specifically, it was a silty sand that bulked very easily and could be placed to a wide range of standard densities, something very necessary to a good test program. The tests confirmed that pipe backfill can be designed, specified, and evaluated on the basis of percent standard AASHTO density, regardless of soil type. The only exceptions are unstable soils, such as those which turn plastic with moisture, even though they have been well compacted to 85% or more standard AASHTO and confined in the fill. Such soils would, of course, not be suitable for a high embankment base, much less for pipe backfill.

Caltrans Tests

A significant research study led by A. E. Bacher of Caltrans in 1975 provided important data on a full scale installation under high fills. This project involved a 10 foot diameter structural plate pipe with a 0.109 inch wall, drastically under-designed to magnify the response and expected to fail. It was loaded with a fill of almost 200 feet, likely the record for this type of test. In addition to demonstrating the remarkable strength of the pipe, measurements of wall stresses and soil pressures contributed to the body of knowledge and gave confidence to design methods used by specifying agencies.

PRODUCT DESIGN PROPERTIES

This section provides properties for the design of all corrugated steel products. Mechanical properties are summarized in Table 7.1 and sectional properties are provided in Table 7.2. Ultimate longitudinal seam strengths are listed in Table 7.3 for riveted CSP, in Table 7.4A for bolted structural plate and in Table 7.4B for deep corrugated plate. Flexibility factors are provided in Table 7.5. The application of this information is discussed subsequently.

Table 7.1

Mechanical properties of products for design

Product	Minimum Yield Point, psi	Minimum Tensile Strength, psi	Minimum Elongation in 2 in.	Modulus of Elasticity, psi
6x2, Type 33	33,000	45,000	25	30,000,000
6x2, Type 38	38,000	48,000	25	30,000,000
15 x 5 1/2 and 16 x 6	44,000	55,000	25	30,000,000
All Other	33,000	45,000	20	30,000,000

Table 7.2

Sectional properties for corrugated steel pipe products

Corrugation in.	0.052	0.064	0.079	0.109 0.111*	0.138 0.140*	0.168 0.170*	0.188	0.218	0.249	0.280	0.310 0.318*	0.389 0.380*
	Specified Thickness, in.											
	Moment of Inertia, I x 10^{-3} (in.^{4}/in.)											
1-1/2 x 1/4	0.343	0.439	0.566	0.857	1.205	1.635						
2 x 1/2	1.533	1.941	2.458	3.541	4.712	5.992						
2-2/3 x 1/2	1.500	1.892	2.392	3.425	4.533	5.725						
3 x 1	6.892	8.658	10.883	15.458	20.175	25.083						
5 x 1		8.850	11.092	15.650	20.317	25.092						
6 x 2				60.41	78.17	96.17	108.00	126.92	146.17	165.83	190.00	232.00
15 x 5 1/2					714.63	874.62	978.64	1143.59	1308.42	1472.17		
3/4 x 3/4x 7 1/2**		2.821	3.701	5.537	7.433							
3/4 x 1x 111/2**		4.580	6.080	9.260								
3/4 x 1x 8 1/2**		5.979	7.913	11.983								
	Area of Wall Cross Section, A (in.^{2}/ft)											
1-1/2 x 1/4	0.608	0.761	0.950	1.331	1.712	2.093						
2 x 1/2	0.652	0.815	1.019	1.428	1.838	2.249						
2-2/3 x 1/2	0.619	0.775	0.968	1.356	1.744	2.133						
3 x 1	0.711	0.890	1.113	1.560	2.008	2.458						
5 x 1		0.794	0.992	1.390	1.788	2.186						
6 x 2				1.556	2.003	2.449	2.739	3.199	3.658	4.119	4.671	5.613
15 x 5 1/2					2.260	2.762	3.088	3.604	4.118	4.633		
3/4 x 3/4x 7 1/2**		0.509	0.712	1.184	1.717							
3/4 x 1x 111/2**		0.374	0.524	0.883								
3/4 x 1x 8 1/2**		0.499	0.694	1.149								
	Radius of Gyration, r (in.)											
1-1/2 x 1/4	0.0824	0.0832	0.0846	0.0879	0.0919	0.0967						
2 x 1/2	0.1682	0.1690	0.1700	0.1725	0.1754	0.1788						
2-2/3 x 1/2	0.1707	0.1712	0.1721	0.1741	0.1766	0.1795						
3 x 1	0.3410	0.3417	0.3427	0.3448	0.3472	0.3499						
5 x 1		0.3657	0.3663	0.3677	0.3693	0.3711						
6 x 2				0.682	0.684	0.686	0.688	0.690	0.692	0.695	0.698	0.704
15 x 5 1/2					1.948	1.949	1.950	1.952	1.953	1.954	1.953	1.954
3/4 x 3/4x 7 1/2**		0.258	0.250	0.237	0.228							
3/4 x 1x 111/2**		0.383	0.373	0.355								
3/4 x 1x 8 1/2**		0.379	0.370	0.354								

* Where two thicknesses are shown, the top value is for corrugated steel pipe and the bottom value is for steel structural plate

** Ribbed pipe; properties are effective values.

For properties of the 16 x 6 in. corrugation, see Table 2.15.

Table 7.3

Ultimate longitudinal seam strength (lbs/ft) for CSP*

CSP Thickness, in.	3 x 1 in.	2-2/3 x 1/2 in. Riveted Seams		
		5/16 in. Single Rivet	5/16 in. Double Rivet	3/8 in. Double Rivet
0.064	28,700 †	16,700 †	21,600	
0.079	35,700 †	18,200 †	29,800	
0.109	53,000		23,400 †	46,800
0.138	63,700 †		24,500 †	49,000 †
0.168	70,700 †		25,600 †	51,300 †

* See Chapter 2 for standard seam details.
† Seams develop full yield strength of pipe wall at 33,000 psi.

Table 7.4A

Ultimate longitudinal seam strength for 6 x 2 in. structural plate*

Structural Plate Thickness, in.	Bolt Diameter, in.	Bolts per ft	Seam Strength, lbs/ft**
0.111	_	4	43,000
0.140	_	4	62,000
0.170	_	4	81,000 †
0.188	_	4	93,000 †
0.218	_	4	112,000 † ‡
0.249	_	4	132,000 † ‡
0.280	_	4	144,000 † ‡
0.280	_	6	180,000 † ‡
0.280	_	8	194,000 † ‡
0.318	7/8	8	235,000 † ‡
0.380	7/8	8	285,000 † ‡

* See Chapter 2 for seam details.
** Industry recognized seam strengths for 6 x 2 in. are published in ASTM A796.
† Seams develop full yield strength of pipe wall at 33,000 psi.
‡ Seams develop full yield strength of pipe wall at 38,000 psi.

Table 7.4B

Ultimate longitudinal seam strength for 15 x 5 1/2 in. deep corrugated*

Structural Plate Thickness, in.	Bolt Diameter, in.	Seam Strength, lbs/ft**
0.140	_	66,000
0.170	_	87,000
0.188	_	102,000
0.218	_	127,000
0.249	_	144,000
0.280	_	144,000
0.249	7/8	159,000
0.280	7/8	177,000

* See Chapter 2 for seam details; 4.8 bolts per foot.
** Industry recognized seam strengths for 15 x 5 1/2 are published in ASTM A796.

Table 7.4C

Ultimate longitudinal seam strength for 16 x 6 in. deep corrugated*

Structural Plate Thickness, in.	Bolt Diameter, in.	Seam Strength, lbs/ft**
0.169	_	81,600
0.197	_	118,900
0.236	_	141,300
0.276	_	153,300
0.315	_	153,300
0.276	7/8	184,000
0.315	7/8	184,000

* See Chapter 2 for seam details; 4.5 bolts per foot.
** At the time this manual went to publication, the design seam strengths for 16 x 6 were not recognized in ASTM A796. Seam strengths shown for 16 x 6 are proprietary values recommended by the manufacturer.

Table 7.5

Recommended limits of Flexibility Factor (FF, in./lb) for round pipe*

2-2/3 x 1/2 Corrugation				
Embankment installations	FF = 0.0433			
Trench installations	FF = 0.0433 for diameters 42 in. or less			
	FF = 0.060 for diameters 48 – 72 in.			
	FF = 0.080 for diameters 78 in. or greater.			
3 x 1 and 5 x 1 Corrugations				
Embankment installations	FF = 0.0433			
Trench installations	FF = 0.060			
Spiral Rib Profiles				
Profile:		3/4 x 3/4 x 7-1/2	3/4 x 1 x 11-1/2	3/4 x 1 x 8-1/2
Type I (embankment) installations:		$0.217\ I^{1/3}$	$0.140\ I^{1/3}$	$0.140\ I^{1/3}$
Type II (trench) installations:		$0.263\ I^{1/3}$	$0.163\ I^{1/3}$	$0.163\ I^{1/3}$
Type III (special trench) installations:		$0.367\ I^{1/3}$	$0.220\ I^{1/3}$	$0.262\ I^{1/3}$
Structural Plate				
6 x 2 corrugation (either trench or embankment):	FF = 0.020			
15 x 5 1/2 corrugation (either trench or embankment):	FF = 0.010			

* Note: For pipe arch, arch, and underpass shapes where rise does not exceed 2/3 of span, the flexibility factor limit is increased: FF = 1.5 FF limit for round pipe.

SOIL CLASSIFICATION SYSTEMS

In selecting soils for backfill, reference is often made to the grouping of soils according to the ASTM United Classification System (UCS) or to the AASHTO M145 system. Table 7.6 provides soil descriptions and a comparison of these systems.

Table 7.6

Soil types by UCS and AASHTO classifications

UCS Soil Classification	AASHTO M 145 Soil Classification		Soil Description
	Group	Subgroup	
	A1		
GW GP SP		A1-a	Well graded gravel
GM SM SP SM		A1-b	Gravelly sand
	A2		
GM SM ML SP GP		A2-4	Sand and gravel with low plasticity silt
SC GC GM		A2-5	Sand and gravels with elastic silt
SC GC		A2-6	Sands with clay fines
SC GC		A2-7	Sands with highly plastic clay fines
SW SP SM	A3		Fine sands, such as beach sand
ML CL OL	A4		Low compressibility silts
MH OH ML OL	A5		High compressibility silts
CL ML CH	A6		Low to medium compressibility silts
OL OH CH CM CL	A7		High compressibility silts and clays
PT OH	A8		Peat and organics; Not suitable as backfill

DESIGN OF STANDARD STRUCTURES

This section presents procedures for the design of standard structures. In this context, standard structures generally refers to round pipe and arches with a maximum span of 26 feet as well as pipe arches and underpasses with a maximum span of 21 feet. Specifically excluded are long span structures, box culverts, and deep corrugated structures.

Backfill Design for Standard Structures

This section discusses backfill design for typical installations. Backfill requirements for long span structures, box culverts and deep corrugated structures are more demanding as treated later in this chapter.

Requirements for selecting and placing backfill material around and near a pipe are similar to those for selecting a roadway embankment fill. The main differences in requirements are due to the fact that the pipe generates more lateral pressure than the earth within the embankment would if no structure existed. Also, the backfill material must be placed and compacted around the pipe without distorting its shape. However, in the end, the quality of the backfill may be dictated by the need to support the pavement over the conduit.

The quality of the backfill is characterized by the soil stiffness, a property that results from the nature of the soil and the level of compaction. See Chapter 10, Installation, for further information on backfill materials and placement. The best backfill materials are non-plastic sands and gravel (GW, GP, GM, SW). Compaction to a minimum density of 90 percent of standard Proctor is generally sufficient.

Often, the backfill for standard structures may be selected from the materials available at the job site. Although highly plastic or organic soils are unsuitable, materials with some degree of plasticity (SM GM, etc.) can be used in most instances. The stiffness of corrugated steel pipe allows these materials to be placed and compacted to the density necessary to support the pipe. AASHTO requires that backfill materials meet AASHTO M 145 requirements for A1, A2 or A3 materials, compacted to 90 percent of standard Proctor density.

The height of final soil cover and the stiffness of the pipe influence the selection of materials. The soil load actually carried by the pipe is affected by the quality (stiffness) of the backfill. Obviously, higher covers dictate better backfill materials. They not only reduce the loads on the pipe, but also provide better support and improve structural strength.

Figure 7.6 By far the most economical choice, this 19 foot diameter, corrugated steel structural plate storm sewer, using crushed rock backfill, carries 90 feet of cover.

As pipes get larger and become more flexible, the choice of materials again becomes more important. The backfill must be compacted sufficiently to provide the necessary pipe support. Well-graded (densely graded), clean, non-plastic materials compact more easily. The reduced compaction forces they require have less effect on the pipe's shape during backfill. These materials also provide more support at a lower density, again reducing the compaction effort required. Because their jagged shape provides a degree of mechanical lock between soil particles, angular materials such as crushed rock typically offer excellent support with relatively minimum compaction effort.

Backfill typically extends to 12 inches above the pipe. A typical specification for pipe backfill under highway pavement may read as follows:

> *Backfill material to a distance of 12 inches above the pipe shall meet the requirements of AASHTO M 145 for A1, A2 or A3 materials. The backfill shall be placed and compacted in 8 to 12 inches loose lift thicknesses to 90% standard Proctor (90% AASHTO T 99) density.*

All state Departments of Transportation have backfill specifications for the installation of CSP under roadways. These specifications recognize local conditions and can provide valuable guidance for the engineer on various pipe projects.

Unlike rigid pipe such as concrete, steel pipe is typically designed to carry the full soil prism above the pipe. There is no concern that excessively wide trenches increase the load on the pipe. On the other hand, it is desirable to minimize trench width to reduce installation cost.

The required trench width, or the minimum backfill width in a normal highway embankment, depends on the backfill material and the compaction equipment used. In trench installations, the backfill must extend from trench wall to trench wall. In sound trench conditions or highway embankment applications, the trench only needs to be wide enough to allow the material to be placed under the haunch and compacted to the specified density. While backfill and trench widths often call for 2 feet on either side of the pipe, crushed stone, flowable gravel and similar soils can be placed in a narrower width.

ASTM A 798, *Standard Practice for Installing Factory-Made Corrugated Steel Pipe for Sewers and Other Installations*, permits the placement of cement slurries or controlled low strength materials with a trench width as little as 6 inches greater than the pipe span. An alternative material, cement stabilized sand, provides excellent support but must be used in a trench width adequate to allow placement and compaction.

With regard to installation demands, the required stiffness of the pipe decreases as the quality of the backfill increases. As subsequently discussed, the design of ribbed pipe takes advantage of this characteristic by defining three different soil conditions, referred to as Type I, Type II, and Type III installations.

Foundation Design for Standard Structures

The supporting soil beneath pipe is generally referred to as the pipe foundation. The foundation under the pipe is not of great concern in most cases. However, standard designs assume the foundation carries the full soil column above the pipe without appreciable settlement. If differential settlement between the pipe and the adjacent backfill does occur, it is desirable for the pipe to settle more than the backfill. This helps to defray any drag down loads that otherwise could occur.

The backfill load on the foundation typically is calculated as the height of the soil column above the pipe, H, times the density, γ, of the backfill and embankment or trench fill above it. Thus, the bearing strength of the foundation should equal or exceed γH. However, pipe arch and underpass shapes require additional considerations. Due to their small radius haunches, these shapes require higher foundation bearing strength levels. Means of determining foundation requirements for these structures are included later in this chapter.

Pipe in a full trench condition generally benefits from a foundation that has been naturally consolidated by the existing soil cover. Where soft foundations are encountered in a trench, they need to be improved by over-excavating and rebuilding the foundation with compacted granular material across the full trench width. Often this consists mostly of removing sloughed material and replacing it with compacted backfill.

Where soft foundations are encountered in embankment conditions, an improved foundation and backfill width equal to one pipe diameter on each side of the pipe is typically specified. This provides a sizeable block of backfill that settles with the pipe and helps ensure adequate pipe support.

Where rock foundations are encountered, it is typical to over-excavate to a maximum of 24 inches and then place 1/2 inch of compacted backfill for each foot of cover between the pipe and the rock.

On either native or improved foundations, a bed of loose material is placed to a minimum thickness of twice the corrugation depth to allow the corrugated pipe wall to nest and become fully supported.

Loads on Standard Structures

The first consideration in structural design is the evaluation of the loads on the pipe. Buried pipe is subject to two principal types of loads:

(1) Dead loads – developed by the embankment or trench fill materials, plus stationary, superimposed surface loads, either uniform or concentrated; and
(2) Live loads – moving loads, including impact, such as from highways, railways, or airplanes.

Dead Loads

The maximum dead load is considered to be the full soil prism over the pipe. The unit pressure of this prism acting on the horizontal plane at the top of pipe is equal to:

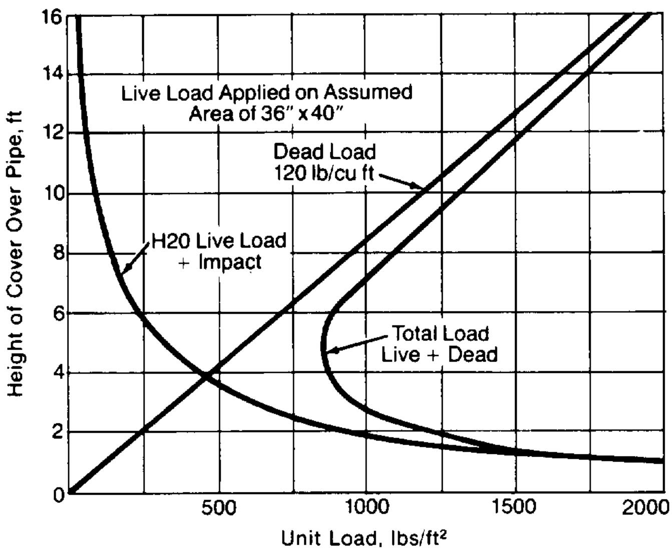
$$DL = \gamma H \tag{1}$$

where

γ = Unit weight of soil, pcf
H = Height of fill over pipe, ft
DL = Dead load pressure, psf

Live Loads

In practice, live loads are typically due to highway, railway, aircraft or construction traffic. Live load pressures on pipe are usually determined from charts initially developed by the corrugated steel pipe industry and adopted by various specifying agencies. Figures 7.7 and 7.8 Show the variation of pressure with depth for a highway and a railway loading. These charts modify the theoretical distribution of live loads to values compatible with observed performance of structures under relatively low covers. Table 7.7 provides tabular values of live load pressure.



■ **Figure 7.7** Combined H20 highway live load and dead load is a minimum at about 5 ft of cover. Live load is applied through a pavement of 1 ft thick.

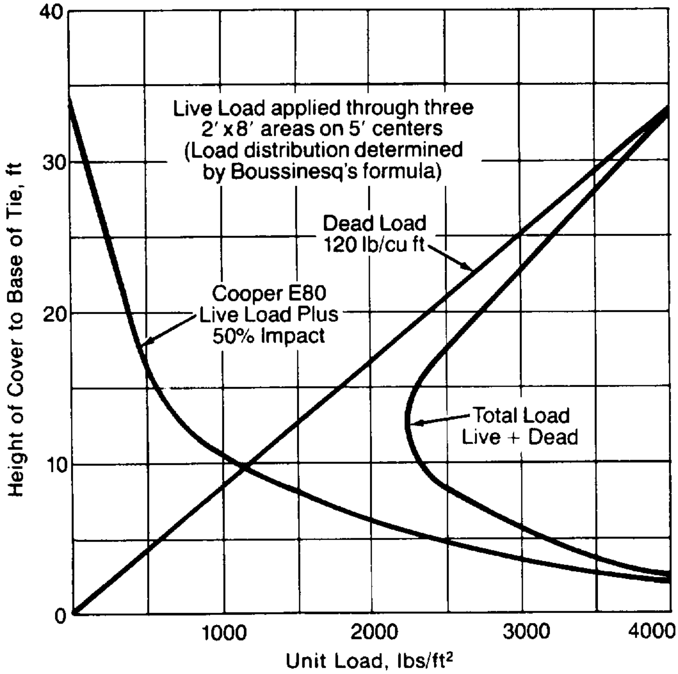


Figure 7.8 Railroad live load, Cooper E 80, combined with dead load is a minimum at about 12 ft. Live load is applied through three 2 ft x 8 ft areas on 5 ft centers.

Table 7.7

Highway and railway live loads (LL)*

Depth of Cover, ft	Highway Loading** Load, psf		Railway E 80 Loading**	
	H20	H25	Depthof Cover, ft	Load psf
1	1800	2280	2	3800
2	800	1150	5	2400
3	600	720	8	1600
4	400	470	10	1100
5	250	330	12	800
6	200	240	15	600
7	175	180	20	300
8	100	140	30	100
9	–	110	–	–

* See ASTM A 796.
** Neglect live load when less than 100 psf; use dead load only.

The live load pressure from other concentrated loads is often calculated on the basis of a load distribution slope of 1/2 to 1 (horizontal to vertical). A method is also provided in the AASHTO *LRFD Bridge Design Specifications*.

Minimum Covers

Minimum covers for H20 and H25 highway loads are taken as the greater of span/8 or 12 inches for all corrugated steel pipe except spiral rib pipe. For spiral rib pipe, this becomes span/4, but not less than 12 inches In all cases, the minimum cover is measured from the top (inside rise) of the pipe to the bottom of the asphalt pavement course and to the top of rigid pavements.

While asphalt does at least as good a job of distributing wheel loads as soil, it is not counted in the minimum cover. The asphalt layer is often very thick and must be placed and compacted in lifts with heavy equipment which would then be on the pipe with inadequate cover. Considering the asphalt thickness as part of the minimum cover could lead to construction problems.

Minimum covers for E 80 railroad loads are twice those for H20 and H25 highway loads, except for structural plate structures. Because of its deeper corrugations and greater bending strength, minimum cover is taken as span/5 or 24 inches, whichever is greater. E 80 minimum covers are measured from the top (inside rise) of the corrugated steel structure to the bottom of the tie.

Guidelines for minimum covers for construction loads are shown in Table 7.8. In some cases the minimum cover provided for design live loads may not be sufficient for the heavier loads from construction equipment. In such cases the construction contractor must provide any additional cover required to avoid damage to the pipe.

Table 7.8

General guidelines for minimum cover required
for heavy off-road construction equipment

Pipe Span, in.	Minimum Cover (ft) for Indicted Axle Loads (kips)*			
	18-50	50-75	75- 110	110- 150
12-42	2.0	2.5	3.0	3.0
48-72	3.0	3.0	3.5	4.0
78-120	3.0	3.5	4.0	4.0
126-144	3.5	4.0	4.5	4.5

* Minimum cover may vary, depending on local conditions. The contractor must provide the additional cover required to avoid damage to the pipe. Minimum cover is measured from the top of the pipe to the top of the maintained construction roadway surface.

The significance of aircraft loads is principally in the area of required minimum cover. Airplanes weighing up to 1-1/4 million pounds and using tire pressures of 225 psi have been used to develop minimum cover tables for the Federal Aviation Administration. See Tables AISI-24 through AISI-27.

Structural Design of Standard Structures by the AISI Method

This section presents a design method for standard structures known as the AISI method. Considerations applicable to standard pipe arches and arches follow in the next section. AASHTO design methods are presented subsequently. As previously stated, standard structures include round pipe and arches with a maximum span of 26 feet as well as pipe arches and underpasses with a maximum span of 21 feet. Specifically excluded are long span structures, box culverts, and deep corrugated structures.

The structural design process consists of the following steps:

1. Select the backfill and other soil densities required or expected.
2. Calculate the design pressure.
3. Compute the compression in the pipe wall.
4. Select the allowable compressive stress.
5. Determine the corrugated steel pipe thickness required.
6. Check minimum handling stiffness.
7. For bolted or riveted pipe only: check seam strength.
8. Pipe arch only: check corner bearing pressure.
9. Arch only: check rise to span ratio (≥ 0.3) and calculate footing reactions.

1. Backfill Density

Select a percent compaction of pipe backfill for design. The value chosen should reflect the height of soil cover on the structure and the backfill quality that reasonably can be expected. The recommended minimum value for routine use in typical installations is 85%. It is good practice to specify a 90% compaction level for installation when 85% is used for design. However, for more important structures under higher cover situations, it is recommended the designer select a higher quality backfill and require the same in construction. This may increase the allowable fill height or save on thickness of the pipe wall.

2. Design Pressure

When the height of cover is equal to or greater than the span or diameter of the structure, enter the load reduction factor from the chart in Figure 7.9, to determine the percentage of the total load acting on the steel. For routine use, the 85% Proctor density value will provide a factor of 0.86. The load reduction factor, K, is applied to the total load to obtain the design pressure, P_v, acting on the steel. If the height of cover is less than one pipe diameter, the total load is assumed to act on the pipe (K = 1). Also, in reclaim (conveyor) tunnel applications, if the ore pile is drawn down and built back up repeatedly, use K = 1.

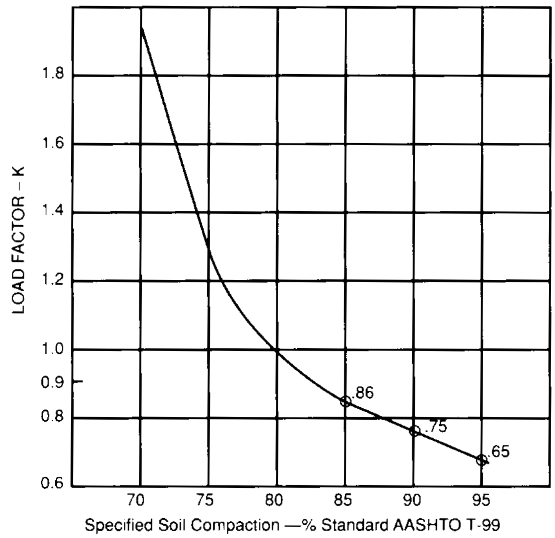


Figure 7.9 Load factors for corrugated steel pipe for backfill compacted to AASHTO T-99 density.

The total load on the pipe becomes:

$$P_v = K\,(DL + LL),\ \text{when } H \geq S \qquad (2a)$$

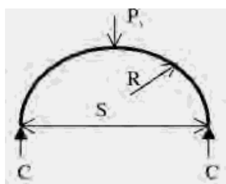
$$P_v = (DL + LL),\ \text{when } H < S \qquad (2b)$$

where
P_v = Design pressure, psf
K = Load reduction factor
DL = Dead load, psf
LL = Live load, psf
H = Height of cover, ft
S = Diameter or span, ft

3. Ring Compression

From fundamental mechanics, the compressive thrust in the conduit wall, *C*, is equal to the radial pressure, *P*, acting on the wall multiplied by the wall radius, *R*, or: $C = PR$. This ring compression thrust, which is the force carried by the steel, acts tangentially to the pipe wall. For conventional structures in which the top arc approaches a semicircle, it is convenient to substitute half the span for the wall radius. Then,

$$C = P_v(S/2) \qquad (3)$$



where

C = Ring compression, lbs/ft
P_v = Vertical design pressure, psf
S = Span, ft

4. Allowable Wall Stress
The ultimate compressive stress (f_b) for corrugated steel structures with a minimum yield point of 33,000 psi and backfill compacted to 85% standard AASHTO density is shown in Figure 7.10. The following gives f_b in equation form:

when D/r ≤ 294

$$f_b = f_y = 33{,}000\text{psi} \tag{4}$$

when 294 < D/r ≤ 500

$$f_b = 40{,}000 - 0.081(D/r)^2 \tag{5}$$

when D/r > 500

$$f_b = \frac{4.93 \times 10^9}{(D/r)^2} \tag{6}$$

where
D = Diameter or span, inches
r = radius of gyration of corrugation (see Table 7.2)

A safety factor of 2 is applied to the ultimate wall stress to obtain the design stress, f_c:

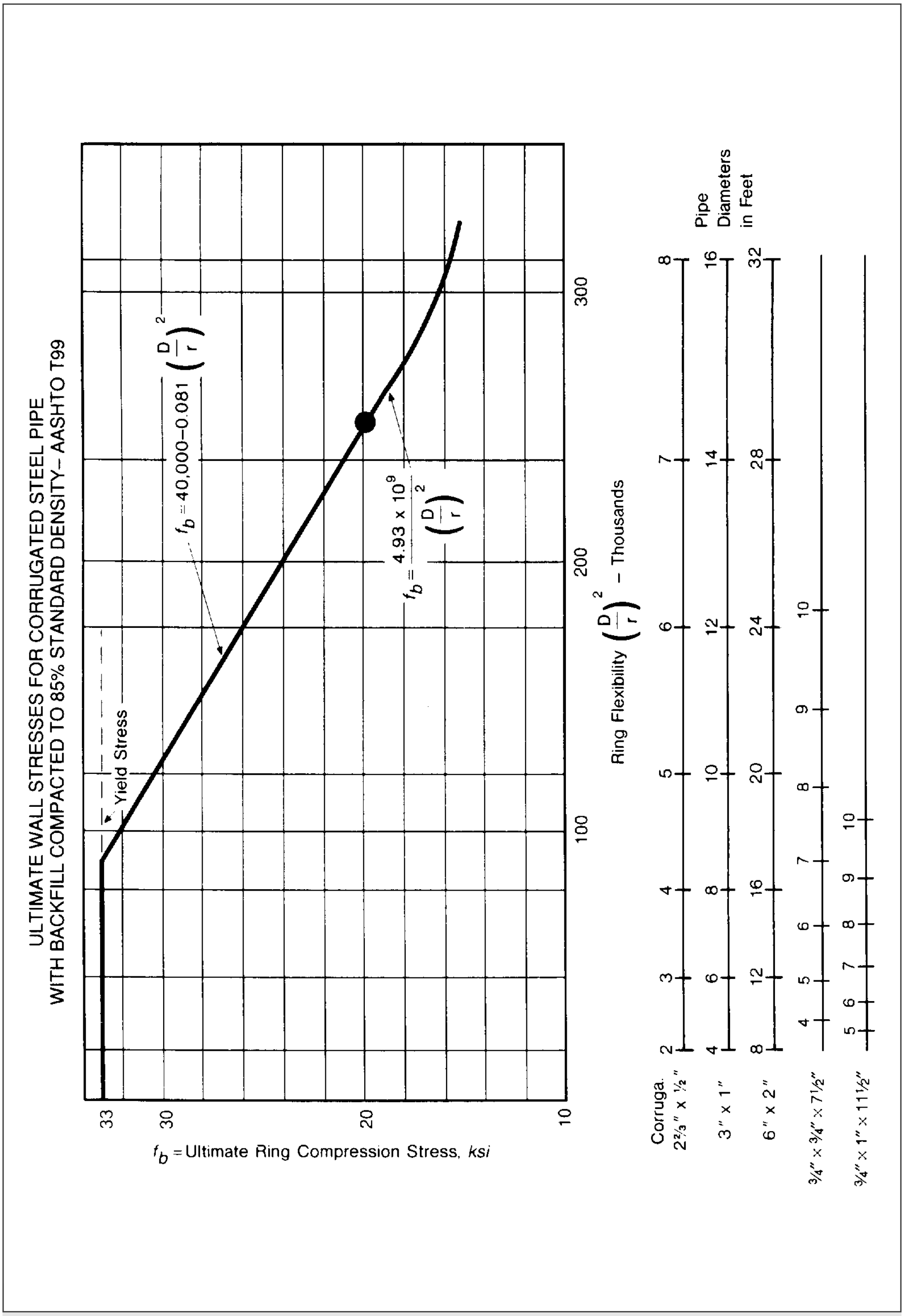
$$f_c = f_b/2 \tag{7}$$

5. Wall Cross-Sectional Area
The required wall area, *A*, is computed from calculated compressive thrust in the pipe wall, *C*, and the allowable stress, f_c.

$$A = C/f_c \tag{8}$$

From Table 7.2 select the wall thickness that provides the required wall area for the same corrugation used to select the allowable stress.

6. Handling Stiffness
Minimum stiffness requirements to assure practical handling and installation without undue care have been established through experience. The resultant flexibility factor, *FF*, limits the size of each combination of corrugation and pipe wall thickness. However, the *FF* limit depends on the type of installation. Embankment installations, which often involve the use of heavier compaction equipment, require a lower *FF* limit (a stiffer pipe) to handle the resulting compaction pressures. Trench installations on the other hand may be designed with a higher *FF* limit (a more flexible pipe) because of the smaller, lighter



■ **Figure 7.10** Ultimate wall or buckling stresses for corrugated steel pipe of various diameters and corrugations. The allowable stress is taken as one-half the ultimate.

compaction equipment employed. The typical, narrow trench does not allow for the use of larger, heavier compaction equipment.

The flexibility factor is expressed as:

$$FF = S^2/EI \text{ in./lb} \tag{9}$$

where

E = Modulus of elasticity of steel = 30,000,000 psi

S = Diameter or span, in.

I = Moment of inertia of corrugation (wall), in.^{4}/in. (see Table 7.2)

Limits for *FF* for round pipe are given in Table 7.5. The note in the table indicates that a 50% increase in flexibility factor limit is allowed for pipe arch, arch, and underpass shapes where the rise does not exceed 2/3 of the span. For these structure shapes, the rise is less than the span. Thus, compared to a round pipe with the same span, there are fewer lifts of backfill that must be placed to get over the structure, and less distortion while the backfill is placed and compacted.

For some pipe arches, fabrication requirements dictate a wall thickness greater than that corresponding to the *FF* limit. Except with plate structures, pipe arches are formed from round pipe and, especially with a 1 inch deep corrugation, a thicker wall may be required for forming. In these instances, the height of cover tables subsequently presented in this chapter show the minimum gage required for fabrication rather than those dictated by the *FF* limit.

For spiral rib pipe, a somewhat different approach is used. To obtain better control, the flexibility factors are varied with corrugation profile, sheet thickness, and type of installation, as shown in Table 7.5. There are three installation types (Type I, II, and III) established for better control. Type I and Type II installations are the traditional embankment and trench installations for all corrugated steel pipe. However, the Type III spiral rib installation goes one step farther, creating a trench installation with special, high quality backfill. These materials – such as crushed rock, pea gravel, cement stabilized sand, etc. – can be compacted to a high strength and stiffness with minimal effort, allowing for proper installation of the more flexible pipe used in Type III installations. The details of the installation requirements are given with the allowable fill heights in Table HC-2.

In the same manner, for pipe of all wall profiles, where special backfill materials or special controls are used, more flexible pipe works well. The use of cementitious grout backfill or controlled low strength materials (CLSM) allows for more flexible pipe than indicated by the trench *FF* limits. They also allow a much narrower trench. In this case, trench widths are limited to the width necessary to place and assemble the pipe. Grout and CLSM flows easily into the pipe haunch area and does not require compaction. Typically a space of only a few inches on each side of the pipe is necessary to place such backfills.

7. Longitudinal Seam Strength
Ultimate longitudinal seam strengths are listed in Table 7.3 for riveted CSP, in Table 7.4A for bolted structural plate, and in Table 7.4B and 7.4C for deep corrugated plate. Seams that develop the full yield strength of the pipe wall are noted. Except for these cases, to maintain a consistent factor of safety of 2.0, it is necessary to limit the maximum ring compression to one half the indicated seam strength.

Expressed in equation form, the required wall seam strength, SS, is calculated from the compressive thrust in the pipe wall, C, using a safety factor of 2.0 as:

$$SS = Cx2 \tag{10}$$

where both C and S have units of lb/ft.

From Table 7.3, 7.4A, or 7.4B, select the wall thickness that provides the required longitudinal seam strength.

Since helical lock seam and continuously welded seam pipe have no longitudinal seams, there is no seam strength check necessary for these types of pipe.

Additional Considerations for Standard Pipe Arch Structures

An additional important design consideration for pipe arches is the corner bearing pressure. Pipe arches generate radial corner pressures as illustrated in Figure 7.11. These haunch pressures, which are greater than the pressure applied at the top of the structure, must be limited to the allowable bearing capacity of the soil adjacent to the haunch. This often becomes the limiting design factor rather than structural strength.

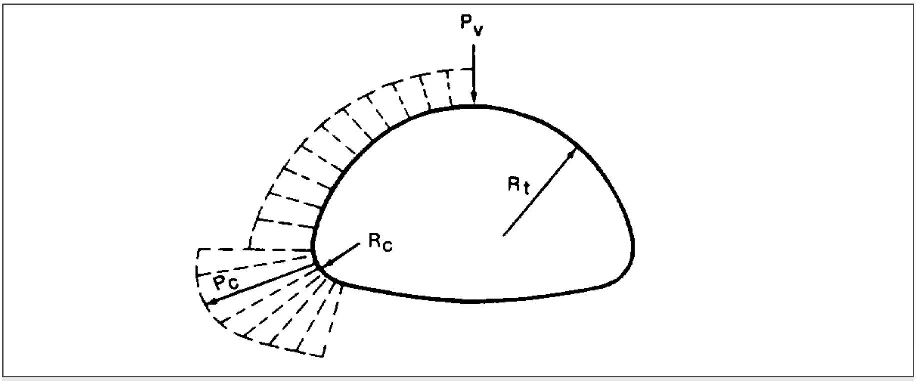


Figure 7.11 The pressure on a pipe arch varies with location and radius, being greatest at the corners.

Dead Load Corner Bearing Pressure. The dead corner pressure can be calculated as follows. Ignoring the bending strength of the pipe wall and the longitudinal distribution of pressure, the ring compression force, *C*, is the same at any point around the structure.

From the familiar relationship $C = P_vR$, the pressure normal to the wall is inversely proportional to the radius. With these assumptions, the corner pressure, P_{CDL}, due to dead (soil) loads would be:

$$P_{CDL} = (R_T/R_C)\, P_{DL} \tag{11}$$

where
- R_T = Top radius, in.
- R_C = Corner (haunch) radius, in.
- P_{DL} = Vertical pressure at top from dead load

This approach calculates the corner pressure at the surface of the pipe. If this bearing pressure is excessive, an extra width of compacted backfill, both beside and below the haunch can be placed to reduce the bearing pressure from that of the pipe arch acting on the trench wall or embankment material. As a simple rule of thumb, extending the backfill a distance of one haunch radius beyond the surface of the haunch reduces pressure on the trench wall or embankment by 50%. A more in-depth evaluation of corner bearing pressures follows.

Live Load Corner Bearing Pressure. The above calculation for P_{CDL} is overly conservative for live loads, such as wheel loads that are not uniformly distributed over the full pipe length. As the ring compression force generated by live loads above the pipe arch is transmitted circumferentially down toward the haunch region, it is also being distributed along the length of the pipe. Thus, the length of the haunch region that transmits the live load pressures into the soil is much greater than the length of pipe arch over which they were initially applied. The corner pressure can be more realistically calculated as:

$$P_{CLL} = R_T C_1 (P_{VLL}/R_C) \tag{12}$$

where
- P_{CLL} = Live load pressure acting on soil at the haunches, psf
- R_T = Radius at crown, in. (≈ 1/2 span; or see tables in Chapter 2)
- C_1 = Longitudinal live load distribution factor
- P_{VLL} = Design live load pressure at crown (psf)
- R_C = Radius at haunch, in. (see tables in Chapter 2)

The total corner bearing pressure then becomes:

$$P_C = P_{CDL} + P_{CLL} \tag{13}$$

This is the procedure that was used to calculate the height-of-cover limits for pipe arches in this design manual. Furthermore, the live load was used without impact because (1) the distance from the point of pressure application to the corner region is greater than the distance from that point to the crown of the structure, and (2) bearing failures are progressive failures occurring over a significant time period as opposed to the brief time of an impact loading. However, the full live load pressure (including impact and unmodified by the C_1 factor) must continue to be used to design the pipe wall.

Equations for C_1, which have been derived from accepted methods, are given below for standard highway and railway loadings. Their derivation is discussed at the end of this section.

C_1 for H20 or H25 highway live loads:
L_1 is the length (inches) over which the live load pressure is applied at the top of the pipe. The length (inches) along the corner which transmits the live load pressure is L_2, when there is no overlap from the wheels at either end of the axle, or L_3, when overlap occurs. Therefore:

$$C_1 = L_1/L_2 \quad \text{when } L_2 < 72 \text{ in.} \tag{14}$$
$$C_1 = 2\,L_1/L_3 \quad \text{when } L_2 > 72 \text{ in.} \tag{15}$$

where

$$L_1 = 40 + (h\text{-}12)1.75 \tag{16}$$
$$L_2 = L_1 + 1.37s \tag{17}$$
$$L_3 = L_2 + 72 \tag{18}$$

h = Height of cover (in.)
s = Span (in)

The live loads for highway loads are as given in Table 7.7 except that the following values (psf) should be used for 1 foot depth of cover:

H20	H25
1600	2000

C_1 for E80 railway live loads:
Because of the function of the tie, there is no pressure overlap for single track arrangements. However, it may be appropriate to consider overlap for some multiple track arrangements. Therefore, for single track arrangements:

$$C_1 = L_1/L_2 \tag{19}$$

where

$$L_1 = 96 + 1.75h \tag{20}$$
$$L_2 = L_1 + 1.37s \tag{21}$$

h = Height of cover (in.)
s = Span (in.)

The live load pressures for railway loads given in Table 7.7 should be divided by 1.5 to remove the impact factor.

Derivation of C_1 for H20 or H25 highway live loads:
The live load pressures for highway live loads have traditionally been based on load application through an assumed 12 inch thick pavement area of 36 by 40 inches Figure 7.12 shows how the load is distributed from an axle load over a pipe arch. The pressure at any height-of-cover h (inches) below the 40 inch wide area is spread over a length L_1 (inches) = 40 + (h – 12)x1.75 at the top of the structure. The stress in the pipe wall from this pressure also spreads longitudinally as it flows toward the corner. Its length at the corner increases by 1.75 times the arc length from the top of the structure to the corner. If this arc length is approximated as (x span/4 = 1.37s) where s is span (inches), it may be seen that the length along the corner which transmits the live load pressure is L_2 (inches) = L_2 + 1.37s where s is the span (inches). No overlap of corner pressure zones occurs until L_2 exceeds 72 inches Thereafter, the reaction length L_3 (inches) = L_2 + 72. Thus, the live load pressure can be multiplied by a coefficient C_1 expressed simply as Equations 14 and 15 above.

Derivation of C_1 for E80 railway live loads:
The live load pressures for each railway axle have traditionally been based on load application through a 24 by 96 inch bearing area. Figure 7.13 shows how the load is distributed from a tie over a pipe arch. The pressure at any height-of-cover h (inches) below the 96 inch tie is spread over a length L_1 (inches) = 96 + 1.75h at the top of the structure. The stress in the pipe wall from this pressure also spreads longitudinally as it flows toward the corner. Its length at the corner increases by 1.75 times the arc length from the top of the structure to the corner. With the same approximations as above, it may be seen that the length along the corner which transmits the live load pressure is L_2 (inches) = L_1 + 1.37s where s is the span (inches). Thus, the live load pressure for a single track railway load can be multiplied by a coefficient C_1 expressed simply as Equation 19 above.

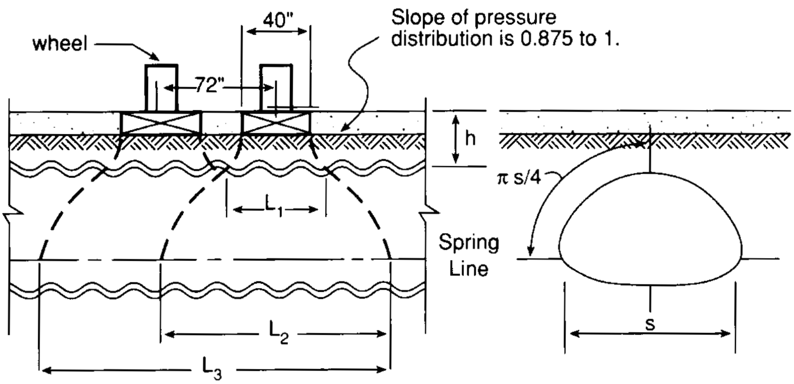
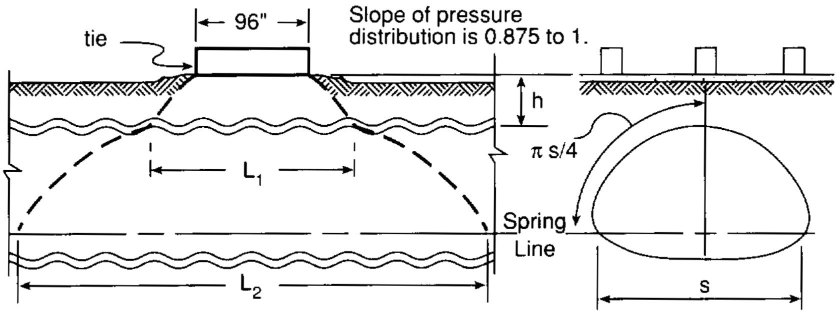


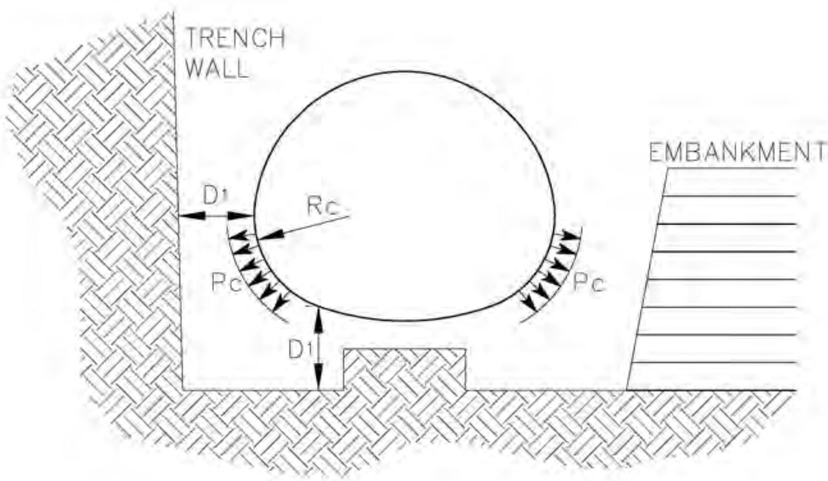
Figure 7.12 Longitudinal distribution of live load corner bearing pressure in pipe-arches under highway loading.



■ **Figure 7.13** Longitudinal distribution of live load corner bearing pressure in pipe-arches under railway loading.

Corner Bearing Pressure at a Distance From the Structure

Where insitu bearing strength conditions dictate, select backfill material can be placed adjacent to the haunch of a pipe arch or other structure. A select granular material is placed and compacted below and beside the haunch in a thickness that allows the bearing pressure at the haunch to spread and dissipate to a level that the insitu material can support (see Figure 7.14).



■ **Figure 7.14** Corner bearing pressure distributed through select, granular fill.

The resulting pressure a distance from the haunch can be calculated as:

$$P_1 = P_c\, R_c / (R_c + D_1) \qquad (22)$$

where

P_1 = Pressure at the desired distance (D_1) from the haunch surface (psf)

P_c = Total dead and live load pressure at the surface of the haunch (psf) (Eq. 13)

R_c = Radius of the haunch (ft)

D_1 = Distance from the haunch (point of interest – ft)

Similarly, the necessary thickness of this select material can be determined from the allowable bearing pressure of the insitu soil as:

$$D_1 = [(R_c\, P_c)/P_{brg}] - R_c) \qquad (23)$$

where

D_1 = Distance from the haunch surface necessary to reduce pressure (ft)

R_c = Radius of the haunch (ft)

P_c = Corner pressure (psf)

P_{brg} = Allowable bearing pressure of the insitu soil (psf)

Additional Considerations for Standard Arch Structures

The design of structural plate arches is based on a minimum ratio of rise to span of 0.3; otherwise, the structural design of the barrel is the same as for structural plate pipe. However, there are two important additional considerations.

The first is foundation rigidity. It is undesirable to make the steel arch relatively unyielding or fixed compared with the adjacent side fill. The use of massive footings or piles to prevent any settlement of the arch is generally not recommended. Where poor materials are encountered, consideration should be given to removing some or all of it and replacing with acceptable material. The footing should be designed to provide uniform, longitudinal settlement of acceptable magnitude from a functional aspect. Allowing the arch to settle will protect it from possible drag-down forces caused by the settlement of the adjacent side fill.

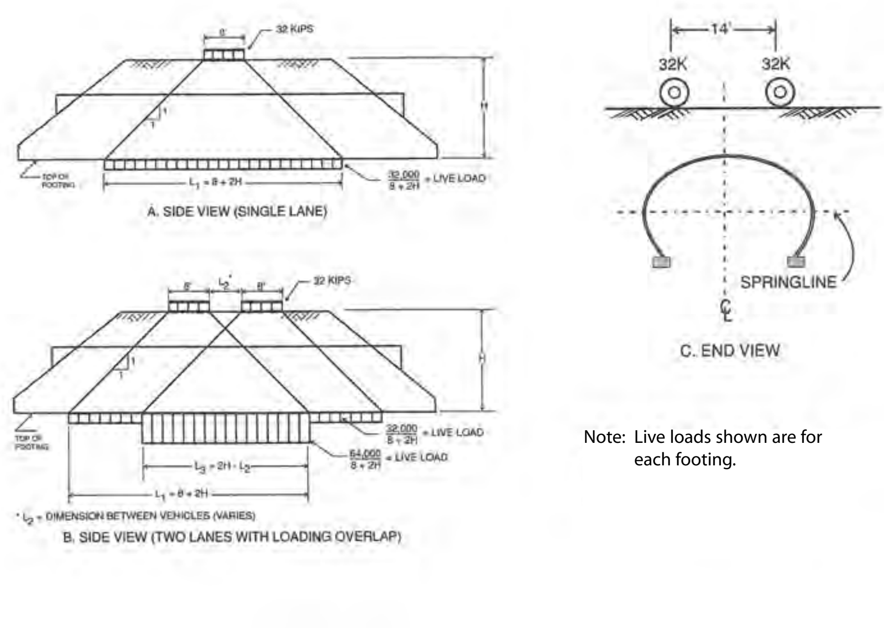
The second consideration is the direction of the forces on the footing. The footing reaction acts tangential to the plate where it connects to the footing. Arches that are not half a circle exert both a vertical and a horizontal reaction on the footing. The value of the tangential footing reaction, which is calculated in a later example, is approximately equal to the thrust in the arch plate at the footing.

However, the vertical footing reaction due to dead (soil) loads can be calculated as follows. Take the vertical dimension from the spring line of the arch to the top of the fill, multiply it by the maximum span of the structure, and then subtract the structure area above the spring line. Multiplied by the density of the soil (usually 120 pcf) to obtain the total soil load on the structure, then divide by two to obtain the vertical soil load on each footing.

Live load footing reactions are calculated as shown in Figures 7.15 and 7.16. The live loads act on the surface and are spread down, through the fill and arch, a distance shown as H, to the elevation of the footing at a 1:1 slope. An H20 wheel load is handled as 64,000 lbs. (80,000 lbs. for H25) applied as two 32,000 lbs. (40,000 lbs. for H25) loads spread over 8 feet on each side of the top centerline of the arch as shown in the figure.

For an H20 live load, the reaction at each footing becomes:

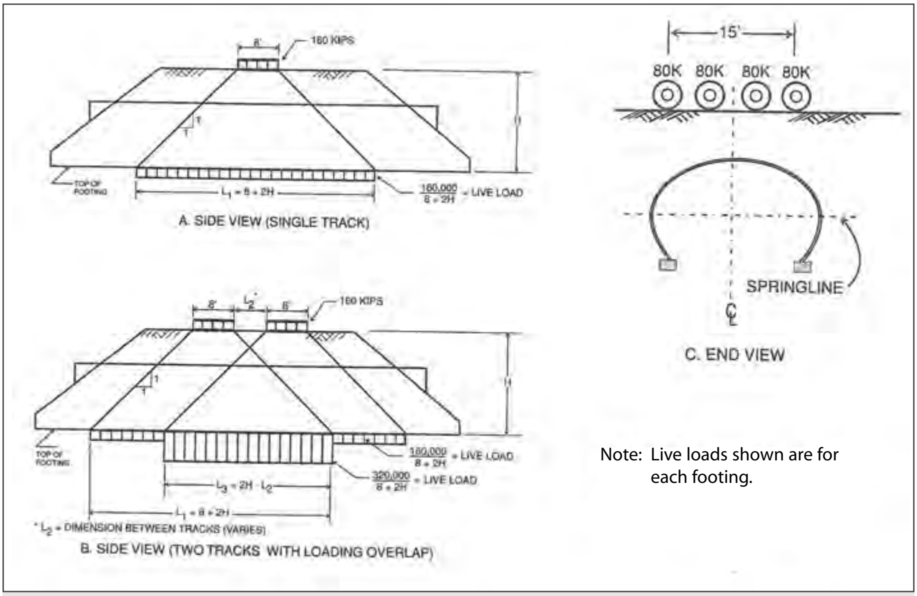
- 32,000/(8+2H) lb/ft, for a single lane crossing.
- 64,000/(8+2H) lb/ft for multiple lanes or meeting vehicles. The length of the overlapping zone, where 64,000/(8+2H) applies, depends on the variable distance between the lanes on the surface and the height H.



■ **Figure 7.15** Details for calculating footing reactions with H20 wheel loads.

Similarly, as shown in Figure 7.16, footing reactions for E80 loads are determined by applying 320,000 lbs. to the fill surface as four 80,000 lbs. concentrated loads on a 5 foot spacing across the span. Each 80,000 lb load is spread over 8 feet longitudinally along the structure by the ties. Thus, the E80 live load reaction at each footing is:

- 160,000 /(8+2H) lb/ft for a single track.
- 320,000/(8+2H) lb/ft for twin tracks. The overlapping zone, where 32,000/(8+2H) depends on the distance between the tacks and the height H.



■ **Figure 7.16** Details for calculating footing reactions with E80 railroad loads.

Deflection Limits for Standard Structures

Early on, designers were concerned about possible excessive pipe deflection levels. Simply stated, corrugated steel pipe is not rigid compared to the clay and concrete pipe of that time. Today it is recognized that excessive deflections are due to inadequate backfill. When backfill materials or their compaction levels are insufficient for the loads, flexible steel pipe will show an unacceptable shape change. It is not feasible to try to control deflection by adjusting the pipe wall stiffness. Instead, deflection is controlled by providing an adequate backfill envelope to support the pipe and its design loads. Strutting CSP is not advised. It is generally ineffective and can result in damaged pipe.

Significant deflection levels in themselves are only an indication that the backfill is consolidating due to the side pressures from the pipe as it seeks support. As the pipe deflects, it moves out and compacts the backfill beside it. In many cases – depending upon the soil type, initial compaction and other factors – this shape change, combined with the overburden pressure, is sufficient to provide the necessary backfill compaction. The structure becomes stable and exhibits the necessary design strength.

CSP is not stress crack sensitive. When the movement stops, if the shape is smooth and exhibits suitable curvature with smooth radius changes, the pipe is usually structurally sound. Bending strains induced by the shape change typically are not detrimental to the performance of the structure or the steel it is made from. After all, corrugated steel pipe is formed by corrugating and curving into the desired shape.

Generally, deflections of 10% of the rise are not considered excessive, provided the shape change has stopped, the shape is suitable for the intended function, and the backfill has become suitably consolidated.

Design Examples for Standard Structures

The following examples illustrate the application of design procedures developed in the preceding pages and referred to as the AISI method. They include: (1) 54 inch diameter pipe under a 60 foot embankment fill, (2) 144 inch diameter pipe in a trench condition, (3) a 20 foot 5 inch pipe arch under 6 feet of cover, and (4) a 23 foot span arch under 19 feet of cover.

Example 1

Given: Pipe diameter, D = S = 54 in.
Seam type: Lock seam – no seam strength check required
Height of cover, H = 60 ft
Live load, LL = H 20 Highway
Weight of soil, γ = 120 pcf
Installation type: Embankment

Find: Wall thickness and type of corrugation.

Solution:

1. Backfill Density:
90% standard Proctor density is specified for construction. Assume a minimum of 85% for design. The height of cover is greater than the span. Therefore, K = 0.86.

2. Design Pressure:
DL = $H\gamma = 60(120) = 7200$ psf
LL = negligible for cover greater than 8 ft (from Table 7.7)
P_v = K(DL + LL)
= 0.86(7200 + 0) = 6190 psf

3. Ring Compression:
   C = $P_v(S/2)$
   = 6190(4.5/2) = 13,900 lbs/ft

4. Allowable Wall Stress:
   Try the 2-2/3 x 1/2 in. corrugation with 0.079 in. wall.
   D/r = 54/0.1721 = 314
   when 294 < D/r ≤ 500, $f_b = 40{,}000 - 0.081(D/r)^2 = 32{,}000$ psi
   $f_c = f_b/2 = 16{,}000$ psi

5. Wall Cross-Sectional Area:
   A = C/f_c = 13,900/16000 = 0.869 in.^{2}/ft required
   From Table 7.2 a specified wall thickness of 0.079 in. provides an uncoated wall area of 0.968 in.^{2}/ft.
   0.869 < 0.968 in.^{2}/ft, **OK**

6. Handling Stiffness:
   FF = S^2/EI
   = $(54)^2/(30{,}000{,}000 \times 0.002392)$
   = 0.0406 in./lb < 0.0433 limit, **OK**

Alternative Solution—Using 3 x 1 in. CSP

4A. Allowable Wall Stress:
   Try the 3 x 1 in. corrugation with 0.064 in. wall.
   D/r = 54/0.3417 = 158
   when D/r < 294, f_b = 33,000 psi
   $f_c = f_b/2 = 16{,}500$ psi

5A. Wall Cross-Sectional Area:
   A = C/f_c = 13,900/16,500 = 0.842 in.^{2}/ft required
   From Table 7.2 a specified thickness of 0.064 in. provides an uncoated wall area of 0.890 in.^{2}/ft.
   0.842 < 0.890 in.^{2}/ft, **OK**

6A. Handling Stiffness:
   FF = S^2/EI
   = $(54)^2/(30{,}000{,}000 \times 0.008658)$
   = 0.0406 in./lb < 0.0433 limit, **OK.**

Results: Acceptable designs include (1.) 2-2/3 x 1/2 inch corrugation with specified wall minimum thickness of 0.079 inch and (2.) 3x1 inch corrugation with specified wall min. thickness of 0.064 inch.

Example 2

Given: Pipe diameter, D = S = 144 in.
Seam type: Lock seam – no seam strength check required
Height of cover, H = 30 ft
Live load, LL = E80 Railway
Weight of soil, γ = 120 pcf
Installation type: Trench

Find: Wall thickness and type of corrugation.

Solution:

1. Backfill Density:
   90% standard Proctor density is specified for construction. Assume a minimum of 85% for design. The height of cover is greater than the span. Therefore, K = 0.86.

2. Design Pressure:
   DL = H γ = 30(120) = 3600 psf
   LL = negligible for cover greater than 8 ft (from Table 7.7)
   P_v = K(DL + LL)
   = 0.86(3600 + 0) = 3100 psf

3. Ring Compression:
   C = P_v(S/2)
   = 3100(12/2) = 18,600 lbs/ft

4. Allowable Wall Stress:
   Try the 5 x 1 in. corrugation with 0.109 in. wall.
   D/r = 144/0.3677 = 392
   when 294 < D/r ≤ 500, f_b = 40,000 - 0.081(D/r)^2 = 27,580 psi
   $f_c = f_b/2$ = 13,790 psi

5. Wall Cross-Sectional Area:
   A = C/f_c= 18,600/13,790 = 1.349 in.^{2}/ft required
   From Table 7.2 a specified thickness of 0.109 in. provides an uncoated wall area of 1.390 in.^{2}/ft.
   1.349 < 1.390 in.^{2}/ft, **OK**

6. Handling Stiffness:
   FF = S^2/EI
   = (144)^2/(30,000,000 x 0.0156)
   = 0.0443 in./lb < 0.060 limit, **OK.**

Results: The 5x1 inch corrugation with specified wall minimum thickness of 0.109 inches is an acceptable design.

Example 3

Given: Structural plate pipe arch with span, S = 20 ft - 5 in. and rise = 13 ft - 0 in.
Corrugation: 6 x 2 in., 31 in corner radius
Height of cover, H = 6 ft
Live load, LL = H20 Highway
Weight of Soil, γ = 120 pcf
Installation type: Trench or embankment

Find: Wall thickness, bolting requirements for longitudinal seams, and corner bearing pressure requirement.

Solution:

1. Backfill Density:
90% standard Proctor density is specified for construction. Assume a minimum of 85% for design. The height of cover is less than the span. Therefore, K = 1.0.

2. Design Pressure:
$DL = H\gamma = 6(120) = 720$ psf
$LL = 200$ psf (from Table 7.7)
$P_v = K(DL + LL)$
$= 1.0(720 + 200) = 920$ psf

3. Ring Compression:
$C = P_v(S/2)$
$= 920(20.42/2) = 9390$ lbs/ft

4. Allowable Wall Stress:
Try the 6 x 2 in. corrugation with 0.140 in. wall.
$D/r = 144/0.684 = 211$
when $D/r < 294$, $f_b = 33{,}000$ psi
$f_c = f_b/2 = 16{,}500$ psi

5. Wall Cross-Sectional Area:
$A = C/f_c = 9390/16{,}500 = 0.569$ in.^{2}/ft required
From Table 7.2 a specified thickness of 0.140 in. provides an uncoated wall area of 2.003 in.^{2}/ft.
$0.569 < 2.003$ in.^{2}/ft, **OK**

Note: A thinner wall would meet this requirement but design is controlled by the flexibility factor limit, step 6.

6. Handling Stiffness:
   FF limit for 6 x 2 in. pipe arch is 0.020 x 1.5 = 0.030 in./lb
   FF = D^2/EI
   = $(245)^2/(30{,}000{,}000 \times 0.07817)$
   = 0.0256 in./lb < 0.030 limit, **OK.**

   Note: A thinner wall would not meet this check.

7. Longitudinal Seam Strength:
   SS = Cx2 = 9390x2 = 18,780 lb/ft required
   From Table 7.4A, the seam strength for 0.140 thickness = 62,000 lbs/ft
   18,780 < 62,000 **OK**

   Note: A thinner wall would meet this requirement but design is controlled by the flexibility factor limit, step 6.

8. Corner Bearing Pressure:
   Calculate C_1:
   $L_1 = 40 + (h-12)1.75 = 40 + (72 - 12)1.75 = 145$ in.
   $L_2 = L_1 + 1.37s = 145 + 1.37(245) = 481$ in. > 72 in.
   $L_3 = L_2 + 72 = 481 + 72 = 553$ in.
   when $L_2 > 72$ in., $C1 = 2L_1/L_3 = 2\text{x}145/553 = 0.524$

   Calculate corner bearing pressure using span/2 for R_T:
   $P_{CLL} = R_T C_1 (P_{VLL}/R_C) = (245/2)(0.524)(200/31) = 414$ psf
   $P_{CDL} = (R_T/R_C)P_{DL} = (122.5/31)720 = 2845$ psf
   $P_C = P_{CDL} + P_{CLL} = 414 + 2845 = 3259$ psf required

   It is imperative that the allowable bearing pressure of the material below and outside the haunch be at least 4000 psf (2 tons/ft^2), which is generally the minimum value used for design.

Results: For the 6 x 2 inch corrugation, a specified wall minimum thickness of 0.140 inch with standard seams (2 bolts/corrugation or 4 bolts/ft) is an acceptable design. Soil in the haunch area must have an allowable bearing pressure of 4000 psf.

Example 4

Given: Structural plate arch with span, S = 23 ft - 0 in. and Rise = 9 ft -10 in
Corrugations: 6 x 2 in.
Height of cover, H = 19 ft
Live load, LL = H20 Highway
Weight of Soil, γ = 120 pcf
Arch return angle (α) is 14.09°
Flow area = 171 ft^2
Installation type: Trench or embankment

Find: Wall thickness, bolting requirements for longitudinal seams, and footing reactions.

Solution:

First check the rise/span ratio: 10.83/23.0 = 0.428 > 0.30. Therefore, structural design is similar to that for round pipe.

1. Backfill Density:
   90% standard Proctor density is specified for construction. Assume a minimum of 85% for design. The height of cover is less than the span. Therefore, K = 1.0.

2. Design Pressure:
   DL = H γ = 19(120) = 2280 psf
   LL = negligible for cover greater than 8 ft (from Table 7.7)
   P_v = K(DL + LL)
   = 1.0(2280 + 0) = 2280 psf

3. Ring Compression:
   C = $P_v(S/2)$
   = 2280(23/2) = 26,220 lbs/ft

4. Allowable Wall Stress:
   Try the 6 x 2 in. corrugation with 0.170 in. wall.
   D/r = 276/0.686 = 402
   when $294 < D/r \leq 500$, $f_b = 40{,}000 - 0.081(D/r)^2 = 26{,}900$ psi
   $f_c = f_b/2 = 13{,}450$ psi

5. Wall Cross-Sectional Area:
   $A = C/f_c$ = 26,220/13,450 = 1.949 in.^{2}/ft required
   From Table 7.2 a specified thickness of 0.170 in. provides an uncoated wall area of 2.449 in.^{2}/ft.
   1.949 < 2.449 in.^{2}/ft, **OK**

   Note: A thinner wall would meet this requirement but design is controlled by the flexibility factor limit, step 6.

6. Handling Stiffness:
   FF limit for 6 x 2 in. pipe arch is 0.020 x 1.5 = 0.030 in./lb
   FF = D^2/EI
   = $(276)^2/(30{,}000{,}000 \times 0.09617)$
   = 0.0264 in./lb < 0.030 limit, **OK.**

   Note: A thinner wall would not meet this check.

7. Longitudinal Seam Strength:
   SS = Cx2 = 26,220x2 = 52,440 lb/ft required
   From Table 7.4A, the seam strength for 0.170 thickness = 81,000 lbs/ft
   52,400 < 81,000 **OK**

   Note: A thinner wall would meet this requirement but design is controlled by the flexibility factor limit, step 6.

8. Footing Reaction
   Weight of soil on arch = [(rise + H) span – flow area]
   = [(9.83 + 19.0) 23 - 171]120 = 59,050 lb/ft.
   R_{dl} = Vertical reaction @ spring line due to soil load
   = Weight of soil/2 = 29,525 lbs/ft
   R_{ll} = Vertical reaction @ spring line due to live load
   Assume two trucks meeting.
   R_{ll} = 64,000/(8 + 2H) = 64,000/{8+[2(19 + 9.83)} = 975 lbs/ft
   $R_{total} = R_{dl} + R_{ll}$ = 29,525 + 975 = 30,500 lbs/ft
   R_v = vertical footing reaction = R cos(α)
   = 30,500 cos(14.09) = 29,580 lbs/ft
   R_h = Horizontal footing reaction = R sin(α)
   = 30,500 sin(14.09) = 7,425 lbs/ft.

Results: For the 6 x 2 inch corrugation, a specified wall min. thickness of 0.170 inchwith standard seams (2 bolts/corrugation or 4 bolts/ft) is an acceptable design. The footings must be designed for R_v = 29,583 lbs/ft and R_h = 7,425 lbs/ft.

HEIGHT OF COVER TABLES FOR STANDARD CORRUGATED STEEL PIPE

The following height-of-cover tables are presented for the designer's convenience to use in routine applications. They are based on the design procedures presented in this chapter for the AISI method. The following values were adopted:

Unit weight of soil = 120 pcf
Density of compacted backfill = 90% AASHTO T-99
AISI load reduction factor K = 0.86

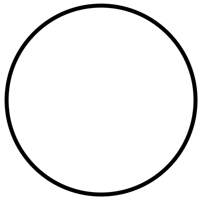
Fill heights for factory made pipe are based on helical seam fabrication. Joint strength must be checked for factory made pipe with other types of seams.

Except as noted, embankment or trench construction is permitted.

List of Height of Cover Tables

	Structure Shape					Live Load		Corrugation Profile				
Table No.	Pipe	Pipe Arch	Horizontal Ellipse	Arch	Under-pass	H20/ H25	E80	2-2/3 x 1/2 in.	5x1 or 3x1 in.	Spiral Rib	6x2 in.	Corner Radius, in.
AISI-1	x					x		x				
AISI-2	x					x				x		
AISI-3	x					x				x		
AISI-4	x					x			x			
AISI-5	x						x	x				
AISI-6	x						x		x			
AISI-7	x					x					x	
AISI-8	x						x				x	
AISI-9		x				x		x				
AISI-10		x				x				x		
AISI-11		x				x			x			
AISI-12		x					x	x				
AISI-13		x					x		x			
AISI-14		x				x					x	18
AISI-15		x				x					x	31
AISI-16		x					x				x	18
AISI-17		x					x				x	31
AISI-18			x			x					x	
AISI-19			x				x				x	
AISI-20					x	x					x	
AISI-21					x		x				x	
AISI-22				x		x					x	
AISI-23				x			x				x	
AISI-24	x					Airport		x				
AISI-25	x					Airport			x			
AISI-26	x					Airport		x	x			x
AISI-27	x					Airport						x

Structural plate pipe used for stockpile tunnels at a copper mine in Utah.



AISI-1

Height of Cover Limits for Steel Pipe
H20 or H25 Live Load • 2-2/3 x 1/2 Corrugation

Diameter or Span, in.	Min.* Cover, in.	Maximum Cover (ft) for Specified Thickness (in.)				
		0.064	0.079	0.109	0.138	0.168
12	12	248	310			
15	12	198	248			
18	12	165	206			
21	12	141	177	248		
24	12	124	155	217		
30	12	99	124	173		
36	12	83	103	145	186	
42	12	71	88	124	159	195
48	12	62	77	108	139	171
54	12	(53)	67	94	122	150
60	12		(57)	80	104	128
66	12			68	88	109
72	12			(57)	75	93
78	12			(48)	63	79
84	12			(40)	52	66
90	12			(32)	43	54
96	12				35	45

Notes:
1. Fill heights in parentheses require standard trench installation; all others may be embankment or trench.
2. In 12 in. through 36 in. diameter, heavier gages may be available – check with the manufacturer.
* Minimum covers are measured from top of pipe to bottom of flexible pavement or top of pipe to top of rigid pavement. Minimum covers must be maintained in unpaved traffic areas.

INSTALLATION AND BACKFILL OF SPIRAL RIB PIPE

Satisfactory backfill material, proper placement, and compaction are key factors in obtaining satisfactory performance.

Minimum pipe metal thickness (gage) is dependent upon minimum & maximum cover and installation TYPE I, II, or III, as noted in the fill height table. Backfill in the pipe envelope shall be granular materials with little or no plasticity; free from rocks, frozen lumps, and foreign matter that could cause hard spots or that could decompose and create voids; compacted to a minimum 90% standard density per ASTM D698 (AASHTO T99).

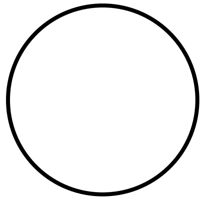
Installation types are:

Type I Installations can be in an embankment or fill condition. Installations shall meet ASTM A798 requirements. ML and CL materials are typically not recommended. Compaction equipment or methods that cause excessive deflection, distortion, or damage shall not be used.

Type II Installations require trench-like conditions where compaction is obtained by hand, or walk behind equipment, or by saturation and vibration. Backfill materials are the same as for TYPE I installations. Special attention should be paid to proper lift thicknesses. Controlled moisture content and uniform gradation of the backfill may be required to limit the compaction effort while maintaining pipe shape.

Type III Installations have the same requirements as TYPE II installations except that backfill materials are limited to clean, non-plastic materials that require little or no compaction effort (GP, SP), or to well graded granular materials classified as GW, SW, GM, SM, GC, or SC with a maximum plasticity index (PI) of 10. Maximum loose lift thickness shall be 8 inches Special attention to moisture content to limit compaction effort may be required. Soil cement or cement slurries may be used in lieu of the selected granular materials.

Note: Simple shape monitoring—measuring the rise and span at several points in the run—is recommended as good practice with all types of installation. It provides a good check on proper backfill placement and compaction methods. Use soil placement and compaction methods which will insure that the vertical pipe dimension (rise) does not increase in excess of 5% of the nominal diameter. Use methods which will insure that the horizontal pipe dimension (span) does not increase in excess of 3% of the nominal diameter. These guidelines will help insure that the final deflections are within normal limits.



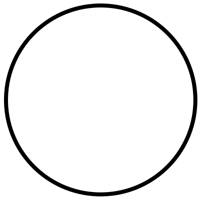
AISI-2

Height of Cover Limits for Spiral Rib Steel Pipe
H20 or H25 Live Load • 3/4 x 3/4 x 7-1/2 in.

Diameter or Span, in.	Min.* Cover, in.	Maximum Cover (ft) for Specified Thickness (in.)			
		0.064	0.079	0.109	0.138
24	12	81	114	189	
30	12	65	91	151	
36	12	54	76	126	
42	12	47	65	108	
48	12	41	57	95	
54	18	(36)	51	84	
60	18	[33]	46	76	110
66	18	[30]	(41)	69	100
72	18		[38]	62	89
78	24		[34]	55	78
84	24			(49)	69
90	24			[43]	61
96	24			[38]	53
102	30			[34]	(47)
108	30				[40]
114	30				[35]
120	30				[30]

Notes:
1. Except as noted, installations may be embankment or trench.
   ( ) Fill heights in parentheses require Type II trench installation.
   [ ] Fill heights in brackets require Type III trench installation.

* Minimum covers are measured from top of pipe to bottom of flexible pavement or top of pipe to top of rigid pavement. Minimum covers must be maintained in unpaved traffic areas.

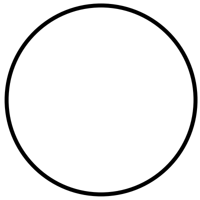


AISI-3

Height of Cover Limits for Spiral Rib Steel Pipe
H20 or H25 Live Load • 3/4 x 1 x 11-1/2 in.

Diameter or Span, in.	Min.* Cover, in.	Maximum Cover (ft) for Specified Thickness (in.)		
		0.064	0.079	0.109
24	12	60	84	141
30	12	48	67	113
36	12	40	56	94
42	12	34	48	81
48	12	30	42	71
54	18	27	37	63
60	18	(24)	34	56
66	18	[22]	30	51
72	18		(28)	47
78	24		[26]	43
84	24		[24]	40
90	24			38
96	24			(35)
102	30			[33]
108	30			[31]

Notes:
1. Except as noted, installations may be embankment or trench.
( ) Fill heights in parentheses require Type II trench installation.
[ ] Fill heights in brackets require Type III trench installation.
* Minimum covers are measured from top of pipe to bottom of flexible pavement or top of pipe to top of rigid pavement. Minimum covers must be maintained in unpaved traffic areas.

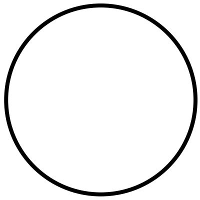


AISI-4

Height of Cover Limits for Steel Pipe
H20 or H25 Live Load • 5 x 1 or 3 x 1 in. Corrugation

Diameter or Span, in.	Min.* Cover, in.	Maximum Cover (ft) for Specified Thickness (in.)				
		0.064	0.079	0.109	0.138	0.168
54	12	56	70	99	127	155
60	12	51	63	89	114	140
66	12	46	58	81	104	127
72	12	42	53	74	95	117
78	12	39	49	68	88	108
84	12	36	45	63	82	100
90	12	34	42	59	76	93
96	12	32	40	56	71	87
102	18	30	37	52	67	82
108	18	(28)	35	49	64	78
114	18	(26)	33	46	59	72
120	18	(24)	30	42	54	67
126	24	(22)	(28)	39	50	62
132	24		(26)	36	47	57
138	24		(24)	33	43	53
144	24			(31)	40	49

Notes:
1. Fill heights in parentheses require standard trench installation; all others may be embankment or trench.
2. Maximum covers shown are for 5 x 1 in.; increase them by 12% for 3 x 1 in.
* Minimum covers are measured from top of pipe to bottom of flexible pavement or top of pipe to top of rigid pavement. Minimum covers must be maintained in unpaved traffic areas.

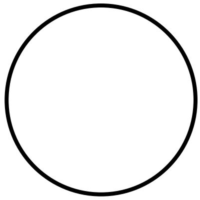


AISI-5

Height of Cover Limits for Steel Pipe
E80 Live Load • 2-2/3 x 1/2 Corrugation

Diameter or Span, in.	Min.* Cover, in.	Maximum Cover (ft) for Specified Thickness (in.)				
		0.064	0.079	0.109	0.138	0.168
12	12	248	310	434		
15	12	198	248	347	446	546
18	12	165	206	289	372	455
21	12	141	177	248	319	390
24	12	124	155	217	279	341
30	12	99	124	173	223	273
36	12	83	103	145	186	227
42	12	71	88	124	159	195
48	12	62	77	108	139	171
54	14		67	94	122	150
60	15			80	104	128
66	17			68	88	109
72	18				75	93
78	20					79
84	21					66

Note:
* From top of pipe to bottom of tie.

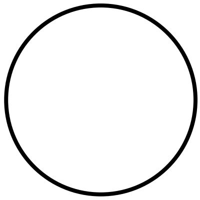


AISI-6

Height of Cover Limits for Steel Pipe
E80 Live Load • 5 x 1 or 3 x 1 in. Corrugation

Diameter or Span, in.	Min.* Cover, in.	Maximum Cover (ft) for Specified Thickness (in.)				
		0.064	0.079	0.109	0.138	0.168
54	18	56	70	99	127	155
60	18	51	63	89	114	140
66	18	46	58	81	104	127
72	18	42	53	74	95	117
78	24	39	49	68	88	108
84	24	36	45	63	82	100
90	24	33**	42	59	76	93
96	24	31**	40	56	71	87
102	30	29**	37	52	67	82
108	30		35	49	64	78
114	30		32**	46	59	72
120	30		30**	42	54	67
126	36			39	50	62
132	36			36	47	57
138	36			33**	43	53
144	36				40	49

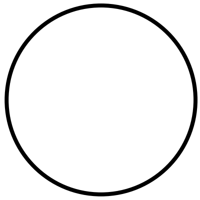
Notes:
1. Maximum covers shown are for 5 x 1 in.; increase them by 12% for 3 x 1 in.
* From top of pipe to bottom of tie.
** These pipe require additional minimum cover.



AISI-7

Height of Cover Limits for Steel Pipe
H25 Live Load • 6 x 2 Corrugation

Diameter or Span,		Min.* Cover,	Maximum Cover (ft) for Specified Thickness (in.)						
ft	in.	in.	0.111	0.140	0.170	0.188	0.218	0.249	0.280
5.0	60	24	81	120	157	175	205	234	263
5.5	66	24	74	109	142	159	186	213	239
6.0	72	24	68	100	131	146	170	195	220
6.5	78	24	63	92	120	135	157	180	203
7.0	84	24	58	86	112	125	146	167	188
7.5	90	24	54	80	104	117	136	156	176
8.0	96	24	51	75	98	109	128	146	165
8.5	102	24	48	71	92	103	120	138	155
9.0	108	24	45	67	87	97	114	130	146
9.5	114	24	43	63	82	92	108	123	139
10.0	120	24	41	60	78	88	102	117	132
10.5	126	30	39	57	75	83	97	111	125
11.0	132	30	37	55	71	80	93	106	120
11.5	138	30	35	52	68	76	89	102	115
12.0	144	30	34	50	65	73	85	97	110
12.5	150	30	33	48	63	70	82	94	105
13.0	156	36	31	46	60	67	79	90	101
13.5	162	36	30	45	58	65	76	87	98
14.0	168	36	29	43	56	63	73	84	94
14.5	174	36	28	41	54	60	71	81	91
15.0	180	36	27	40	52	58	68	78	88
15.5	186	42	26	39	51	57	66	75	85
16.0	192	42		38	49	55	64	73	82
16.5	198	42		36	47	53	62	71	80
17.0	204	42		35	46	51	60	69	77
17.5	210	42		34	44	49	58	66	74
18.0	216	48		33	42	47	55	63	71
18.5	222	48			40	45	53	61	68
19.0	228	48			39	43	51	58	66
19.5	234	48			37	42	49	56	63
20.0	240	48			36	40	47	54	61
20.5	246	54				38	45	51	58
21.0	252	54				37	43	49	56
21.5	258	54					41	47	54
22.0	264	54					40	45	51
22.5	270	60					38	44	49
23.0	276	60						42	47
23.5	282	60						40	45
24.0	288	60						38	43
24.5	294	60						37	42
25.0	300	60							40
25.5	306	60							38
26.0	312	60							36

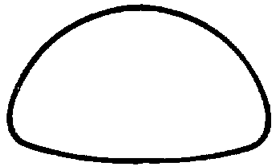


AISI-8

Height of Cover Limits for Steel Pipe
E80 Live Load • 6 x 2 Corrugation

Diameter or Span,		Min.* Cover,	Maximum Cover (ft) for Specified Thickness (in.)						
ft	in.	in.	0.111	0.140	0.170	0.188	0.218	0.249	0.280
5.0	60	24	81	120	157	175	205	234	263
5.5	66	24	74	109	142	159	186	213	239
6.0	72	24	68	100	131	146	170	195	220
6.5	78	24	63	92	120	135	157	180	203
7.0	84	24	58	86	112	125	146	167	188
7.5	90	24	54	80	104	117	136	156	176
8.0	96	24	51	75	98	109	128	146	165
8.5	102	24	48	71	92	103	120	138	155
9.0	108	24	45	67	87	97	114	130	146
9.5	114	24	43	63	82	92	108	123	139
10.0	120	24	41	60	78	88	102	117	132
10.5	126	30	39	57	75	83	97	111	125
11.0	132	30	37	55	71	80	93	106	120
11.5	138	30	35**	52	68	76	89	102	115
12.0	144	30	34**	50	65	73	85	97	110
12.5	150	30	32**	48	63	70	82	94	105
13.0	156	36	31**	46	60	67	79	90	101
13.5	162	36	29**	45	58	65	76	87	98
14.0	168	36	28**	43	56	63	73	84	94
14.5	174	36	26**	41	54	60	71	81	91
15.0	180	36	25**	40	52	58	68	78	88
15.5	186	42	24**	39	51	57	66	75	85
16.0	192	42	23**	38	49	55	64	73	82
16.5	198	42		36	47	53	62	71	80
17.0	204	42		35	46	51	60	69	77
17.5	210	42		34	44	49	58	66	74
18.0	216	48		33	42	47	55	63	71
18.5	222	48			40	45	53	61	68
19.0	228	48			39	43	51	58	66
19.5	234	48			37	42	49	56	63
20.0	240	48			36	40	47	54	61
20.5	246	54				38	45	51	58
21.0	252	54				37	43	49	56
21.5	258	54					41	47	54
22.0	264	54					40	45	51
22.5	270	60					38	44	49
23.0	276	60						42	47
23.5	282	60						40	45
24.0	288	60						38	43
24.5	294	60						37	42
25.0	300	60							40
25.5	306	60							38
26.0	312	60							36

Notes:
* From top of pipe to bottom of tie.
** These pipe require additional minimum cover.



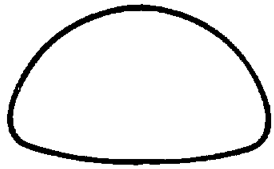
AISI-9

Height-of-Cover Limits for Corrugated Steel Pipe Arch
H20 or H25 Live Load • 2-2/3 x 1/2 in. Corrugation

Span & Rise in.	Minimum Specified Thickness Required in.	Maximum Cover (ft) Over Pipe Arch for Soil Corner Bearing Capacity of 2 tons/ft^2	Diameter or Span in.
17 x 13	0.064	16	12
21 x 15	0.064	15	12
24 x l 8	0.064	15	12
28 x 20	0.064	15	12
35 x 24	0.064	15	12
42 x 29	0.064	15	12
49 x 33	0.079	15	12
57 x 38	0.109	15	12
64 x 43	0.1 09	15	12
71 x 47	0.138	15	12
77 x 52	0.168	15	12
83 x 57	0.168	15	12

Notes:
1. Soil bearing capacity refers to the soil in the region of the pipe corners. See Chapter 10 for design of pipe envelope at pipe corners. The remaining backfill around the pipe arch must be compacted to a specified AASHTO T-99 density of 90%.
2. Use reasonable care in handling and installation.

* Minimum covers are for H20 and H25 loads. See Table 10.1 for construction load requirements. Minimum covers are measured from top of pipe to bottom of flexible pave-ment or top of pipe to top of rigid pavement. Minimum cover must be maintained in unpaved traffic areas.

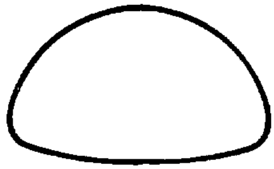


AISI-10

Height-of-Cover Limits for Steel Spiral Rib Pipe Arch
H20 or H25 Live Load • 3/4 x 3/4 x 71/2 in. and 3/4 x 1 x 111/2 in. Configurations

Span & Rise in.	Minimum Specified Thickness Required in.	Minimum Cover in.	Maximum Cover (ft) Over Pipe Arch for Soil Corner Bearing Capacity of 2 tons/ft^2
20 x 16	0.064	12	13
23 x 19	0.064	12	14
27 x 21	0.064	12	13
33 x 26	0.064	12	13
40 x 31	0.064	12	13
46 x 36	0.064	12	14
53 x 41	0.064	18	(13)
60 x 46	0.079	18	20
66 x 51	0.079	18	(21)
73 x 55	0.109	18	21
81 x 59	0.109	18	(17)
87 x 63	0.109	18	(17)
95 x 67	0.109	18	(17)

Notes:
l . Soil bearing capacity refers to the soil in the region of the pipe corners. See Chapter 10.
2. Minimum covers are for H20 and H25 loads. See Table 10.1 for heavy construction load requirements. Minimum covers are measured from top of pipe to bottom of flexible pavement or top of pipe to top of rigid pavement. Minimum cover must be maintained in unpaved traffic areas.
3. TYPE l installations are allowed unless otherwise shown.
4. ( ) Requires TYPE II installation
5. [ ] Requires TYPE III installation
6. For more details on TYPE I, II, and III installations, refer to the section on Installation and Backfill of Spiral Rib Pipe found earlier in this chapter.



AISI-11

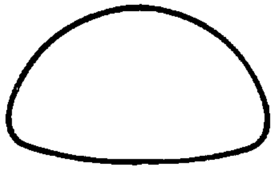
Height-of-Cover Limits for Corrugated Steel Pipe Arch
H20 or H25 Live Load • 5 x 1 in. and 3 x 1 in. Corrugations

Span & Rise in.	Minimum Specified Thickness Required 3 x 1 in.	5 x 1** in.	Minimum* Cover in.	Maximum Cover (ft) Over Pipe Arch for Soil Corner Bearing Capacity of 2 tons/ft^2
53 x 41	0.079	0.109	12	25
60 x 46	0.079	0.109	15	25
66 x 51	0.079	0.109	15	25
73 x 55	0.079	0.109	18	24
81 x59	0.079	0.109	18	21
87 x 63	0.079	0.109	18	20
95 x 67	0.079	0.109	18	20
103 x 71	0.079	0.109	18	20
112 x 75	0.079	0.109	21	20
117 x 79	0.109	0.109	21	19
128 x 83	0.109	0.109	24	19
137 x 87	0.109	0.109	24	19
142 x 91	0.138	0.138	24	19
150 x 96	0.138	0.138	30	19
157 x 101	0.138	0.138	30	19
164 x 105	0.138	0.138	30	19
171 x 110	0.138	0.138	30	19

Notes:
1. Soil bearing capacity refers to the soil in the region of the pipe corners. See Chapter 10 for design of pipe envelope at pipe corners. The remaining backfill around the pipe arch must be compacted to a specified AASHTO T-99 density of 90%
2. Use reasonable care in handling and installation.
3. Pipe arches are typically used where the cover does not exceed 15 feet.

* Minimum covers are for H20 and H25 loads. See Table 10.1 for construction load requirements. Minimum covers are measured from top of pipe to bottom of flexible pave-ment or top of pipe to top of rigid pavement. Minimum cover must be maintained in unpaved traffic areas.

** Same thicknesses as specified for 3 x 1 may be provided when the corner radius meets the requirements of ASTM A760.



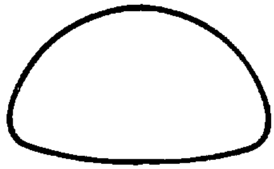
AISI-12

Height-of-Cover for Corrugated Steel Pipe Arch
E80 Live Load •2 2/3 x 1/2 in. Corrugation

Span & Rise in.	Minimum Specified Thickness Required in.	Minimum* Cover in.	Maximum Cover (ft) Over Pipe Arch for Soil Corner Bearing Capacity of 3 tons/ft^2
17 x 13	0.079	24	22
21 x 15	0.079	24	22
24 x 18	0.109	24	22
28 x 20	0.109	24	22
35 x 24	0.138	24	22
42 x 29	0.1038	24	22
49 x 33	0.168	24	22
57 x 38	0.168	24	22
64 x 43	0.168	24	22
71 x47	0.168	24	22

Notes:
1. Soil bearing capacity refers to the soil in the region of the pipe corners. See Chapter 10 for design of pipe envelope at pipe corners. The remaining backfill around the pipe arch must be compacted to a specified AASHTO T-99 density of 90%
2. Use reasonable care in handling and installation.
3. Pipe arches are typically used where the cover does not exceed 15 feet.

* Minimum cover is from top of pipe to bottom of tie.



AISI-13

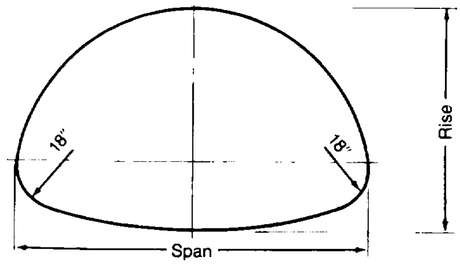
Height-of-Cover for Corrugated Steel Pipe Arch
E80 Live Load • 5 x 1 in. and 3 x 1 in. Corrugations

Span & Rise in.	Minimum Specified Thickness Required 3 x 1 in.	Minimum Specified Thickness Required 5 x 1** in.	Minimum* Cover in.	Maximum Cover (ft) Over Pipe Arch for Soil Corner Bearing Capacity of 2 tons/ft^2
53 x 41	0.079	0.109	24	25
60 x 46	0.079	0.109	24	25
66 x 51	0.079	0.109	24	25
73 x 55	0.079	0.109	30	24
81 x 59	0.079	0.109	30	21
87 x 63	0.079	0.109	30	18
95 x 67	0.079	0.109	30	18
103 x 71	0.079	0.109	36	18
112 x 75	0.079	0.109	36	18
117 x 79	0.109	0.109	36	17
128 x 83	0.109	0.109	42	17
137 x 87	0.109	0.109	42	17
142 x 91	0.138	0.138	42	17
150 x 96	0.138	0.138	48	17
157 x 101	0.138	0.138	48	17
164 x 105	0.138	0.138	48	17
171 x 110	0.138	0.138	48	17

Notes:
1. Soil bearing capacity refers to the soil in the region of the pipe corners. See Chapter 10 for design of pipe envelope at pipe corners. The remaining backfill around the pipe arch must be compacted to a specified AASHTO T-99 density of 90%.
2. Use reasonable care in handling and installation.
3. Pipe arches are typically used where the cover does not exceed 15 feet.

* From top of pipe to bottom of tie.

** Lesser thicknesses may be provided if justified by calculations.

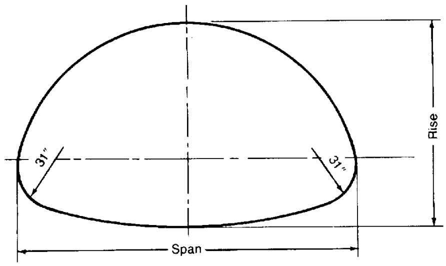


AISI-14

Height-of-Cover Limits for Structural Plate Pipe Arch • 18 in. R_C Corner Radius
H20 or H25 Live Load • 6 x 2 in. Corrugation

Size		Minimum Specified Thickness Required	Minimum* Cover	Maximum Cover (ft) Over Pipe Arch for the Following Soil Corner Bearing Capacities	
Span ft-in.	Rise ft-in.	in.	in.	2 tons/ft^2	3 tons/ft^2
6-1	4-7	0.111	12	19	
6-4	4 9	0.111	12	18	
6-9	4-11	0.111	12	17	
7-0	5-1	0.111	12	16	
7-3	5-3	0.111	12	16	
7-8	5-5	0.111	12	15	
7-11	5-7	0.111	12	14	
8-2	5-9	0.111	18	14	
8-7	5-11	0.111	18	13	
8-10	6-1	0.111	18	13	
9-4	6-3	0.111	18	12	
9-6	6-5	0.111	18	12	
9-9	6-7	0.111	18	12	
10-3	6-9	0.111	18	10	
10-8	6-11	0.111	18	8	
10-11	7-1	0.111	18	8	
11-5	7-3	0.111	18	8	15
11-7	7-5	0.111	18	8	15
11-10	7-7	0.111	18	7	14
12-4	7-9	0.111	24	6	12
12-6	7-11	0.111	24	6	12
12-8	8-1	0.111	24	6	11
12-10	8-4	0.111	24	6	11
13-5	8-5	0.111	24	5	11
13-11	8-7	0.111	24	5	10
14-1	8-9	0.111	24	5	10
14-3	8-11	0.111	24	5	10
14-10	9-1	0.111	24	5	10
15-4	9-3	0.111	24		9
15-6	9-5	0.111	24		9
15-8	9-7	0.111	24		9
15-10	9-10	0.111	24		9
16-5	9-11	0.111	30		9
16-7	10-1	0.111	30		9

Notes:
1. Soil bearing capacity refers to the soil in the region of the pipe corners. See Chapter 10 for design of pipe envelope at pipe corners. The remaining backfill around the pipe arch must be compacted to a specified AASHTO T-99 density of 90%.
2. Use reasonable care in handling and installation.
3. Pipe arches are typically used where the cover does not exceed 15 feet.
* Minimum covers are for H20 and H25 loads. See Table 7.8 for construction load requirements. Minimum covers are measured from top of pipe to bottom of flexible pave-ment or top of pipe to top of rigid pavement. Minimum cover must be maintained in unpaved traffic areas.



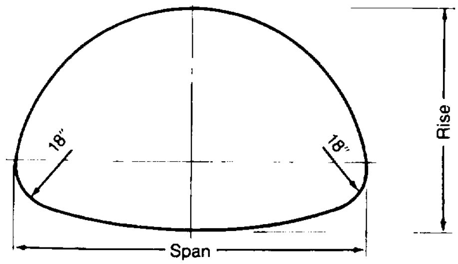
AISI-15

Height-of-Cover Limits for Structural Plate Pipe Arch • 31 in. R_C Corner Radius
H20 or H25 Live Load • 6 x 2 in. Corrugation

Size		Minimum Specified Thickness Required in.	Minimum* Cover in.	Maximum Cover (ft) Over Pipe Arch for the Following Soil Corner Bearing Capacities	
Span ft-in.	Rise ft-in.			2 tons/ft^2	3 tons/ft^2
13-3	9-4	0.111	24	13	
13-6	9-6	0.111	24	13	
14-0	9-8	0.111	24	12	
14-2	9-10	0.111	24	12	
14-5	10-0	0.111	24	12	
14-11	10-2	0.111	24	12	
15-4	10-4	0.111	24	11	
15-7	10-6	0.111	24	11	
15-10	10-8	0.111	24	10	
16-3	10-10	0.111	30	10	
16-6	11-0	0.111	30	10	
17-0	11-2	0.111	30	10	15
17-2	11-4	0.111	30	10	15
17-5	11-6	0.111	30	10	15
17-11	11-8	0.111	30	10	14
18-1	11-10	0.111	30	9	14
18-7	12-0	0.111	30	9	14
18-9	12-2	0.111	30	9	14
19-3	12-4	0.111	30	9	13
19-6	12-6	0.140	30	9	13
19-8	12-8	0.140	30	9	13
19-11	12-10	0.140	30	9	13
20-5	13-0	0.140	36	8	13
20-7	13-2	0.140	36	8	13

Notes:
1. Soil bearing capacity refers to the soil in the region of the pipe corners. See Chapter 10 for design of pipe envelope at pipe corners. The remaining backfill around the pipe arch must be compacted to a specified AASHTO T-99 density of 90%.
2. Use reasonable care in handling and installation.

* Minimum covers are for H20 and H25 loads. See Table 7.8 for construction load requirements. Minimum covers are measured from top of pipe to bottom of flexible pave-ment or top of pipe to top of rigid pavement. Minimum cover must be maintained in unpaved traffic areas.

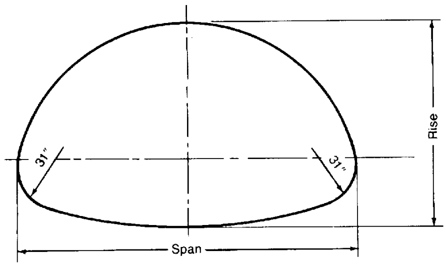


AISI-16

Height-of-Cover Limits for Structural Plate Pipe Arch • 18 in. R_C Corner Radius
E80 Live Load • 6 x 2 in. Corrugation

Size		Minimum Specified Thickness Required in.	Minimum* Cover in.	Maximum Cover (ft) Over Pipe Arch for the Following Soil Corner Bearing Capacities		
Span ft-in.	Rise ft-in.			2 tons/ft^2	3 tons/ft^2	4 tons/ft^2
6-1	4-7	0.111	24	19		
6-4	4-9	0.111	24	15		
6-9	4-11	0.111	24	15		
7-0	5-1	0.111	24	13		
7-3	5-3	0.111	24	12		
7-8	5-5	0.111	24	12		
7-11	5-7	0.111	24	11		
8-2	5-9	0.111	24	10		
8-7	5-11	0.111	24	6		
8-10	6-1	0.111	24	5		
9-4	6-3	0.111	24		17	
9-6	6-5	0.111	24		16	
9-9	6-7	0.111	24		16	
10-3	6-9	0.111	30		15	
10-8	6-11	0.111	30		13	
10-11	7-1	0.111	30		13	
11-5	7-3	0.111	30		12	
11-7	7-5	0.140	30		12	
11-10	7-7	0.140	30		12	
12-4	7-9	0.140	30		6	
12-6	7-11	0.140	30		6	16
12-8	8-1	0.140	36		6	16
12-10	8-4	0.140	36		6	16
13-5	8-5	0.140	36			15
13-11	8-7	0.140	36			15
14-1	8-9	0.140	36			14
14-3	8-11	0.140	36			11
14-10	9-1	0.140	36			9
15-4	9-3	0.140	42			9
15-6	9-5	0.140	42			9
15-8	9-7	0.140	42			9
15-10	9-10	0.140	42			9
16-5	9-11	0.140	42			7
16-7	10-1	0.140	42			7

Notes:
1. Soil bearing capacity refers to the soil in the region of the pipe corners. See Chapter 10 for design of pipe envelope at pipe corners. The remaining backfill around the pipe arch must be compacted to a specified AASHTO T-99 density of 90%.
2. Use reasonable care in handling and installation.
3. Pipe arches are typically used where the cover does not exceed 15 feet.
* From top of pipe to bottom of tie.



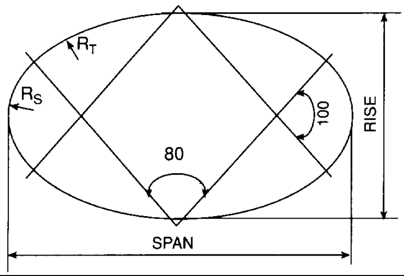
AISI-17

Height-of-Cover Limits for Structural Plate Pipe Arch • 31 in. R_C Corner Radius
E80 Live Load • 6 x 2 in. Corrugation

Size		Minimum Specified Thickness Required in.	Minimum* Cover in.	Maximum Cover (ft) Over Pipe Arch for the Following Soil Corner Bearing Capacities	
Span ft-in.	Rise ft-in.			2 tons/ft²	3 tons/ft²
13-3	9-4	0.140	36	9	22
13-6	9-6	0.140	36	8	22
14-0	9-8	0.140	36	6	21
14-2	9-10	0.140	36	6	21
14-5	10-0	0.140	36	6	21
14-11	10-2	0.140	36	6	20
15-4	10-4	0.140	42	6	19
15-7	10-6	0.140	42	6	19
15-10	10-8	0.140	42	6	19
16-3	10-10	0.140	42		14
16-6	11-0	0.140	42		14
17-0	11-2	0.140	42		13
17-2	11-4	0.140	42		13
17-5	11-6	0.140	42		
17-11	11-8	0.140	48		11
18-1	11-10	0.140	48		11
18-7	12-0	0.140	48		11
18-9	12-2	0.140	48		11
19-3	12-4	0.140	48		10
19-6	12-6	0.170	48		10
19-8	12-8	0.170	48		10
19-11	12-10	0.170	48		10
20-5	13-0	0.170	48		10
20-7	13-2	0.170	48		10

Notes:
1. Soil bearing capacity refers to the soil in the region of the pipe corners. See Chapter 10 for design of pipe envelope at pipe corners. The remaining backfill around the pipe arch must be compacted to a specified AASHTO T-99 density of 90%.
2. Use reasonable care in handling and installation.
3. Pipe arches are typically used where the cover does not exceed 15 feet.

*From top of pipe to bottom of tie.

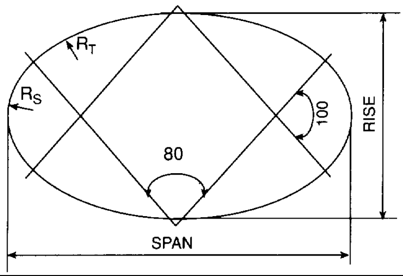


AISI-18

Height-of-Cover Limits for Structural Plate Horizontal Elliptical Pipe
H20 or H25 Live Load • 6 x 2 in. Corrugation

Pipe Size	Span ft-in.	Rise ft-in.	R_T in.	R_s in.	Minimum* Cover in.	Minimum Specified Thickness Required in.	Maximum Cover (ft) Over Pipe for Side and Haunch Soil Bearing Capacity of 2 tons/ft²
24 E 15	7-4	5-6	54.00	26.50	12	0.111	16
27 E 15	8-1	5-9	60.88	26.50	18	0.111	14
30 E 15	8-10	6-0	67.75	26.50	18	0.111	13
30 E 18	9-2	6-9	67.75	32.00	18	0.111	15
33 E 15	9-7	6-4	74.63	26.50	18	0.111	11
33 E 18	9-11	7-0	74.63	32.00	18	0.111	14
36 E 15	10-4	6-7	81.51	26.50	18	0.111	10
36 E 18	10-8	7-3	81.51	32.00	18	0.111	13
36 E 21	11-0	8-0	81.51	37.50	18	0.111	15
39 E 15	11-1	6-10	88.38	26.50	18	0.111	10
39 E 18	11-4	7-6	88.38	32.00	18	0.111	12
39 E 21	11-8	8-3	88.38	37.50	18	0.111	14
39 E 24	12-0	8-11	88.38	43.00	24	0.111	16
42 E 15	11-9	7-1	95.26	26.50	18	0.111	9
42 E 18	12-1	7-10	95.26	32.00	24	0.111	11
42 E 21	12-5	8-6	95.26	37.50	24	0.111	13
42 E 24	12-9	9-2	95.26	43.00	24	0.111	15
45 E 15	12-6	7-4	102.13	26.50	24	0.111	8
45 E 18	12-10	8-1	102.13	32.00	24	0.111	10
45 E 21	13-2	8-9	102.13	37.50	24	0.111	12
45 E 24	13-6	9-6	102.13	43.00	24	0.111	14
48 E 18	13-7	8-4	109.01	32.00	24	0.111	9
48 E 21	13-11	9-0	109.01	37.50	24	0.111	11
48 E 24	14-3	9-9	109.01	43.00	24	0.111	13
48 E 27	41-7	10-5	109.01	48.50	24	0.111	14
48 E 30	14-11	11-2	109.01	54.00	24	0.111	16

Notes:
1. Soil bearing capacity refers to the soil in the region of the pipe haunches. See Chapter 10 for design of pipe envelope at pipe haunches. The remaining backfill around the ellipse must be compacted to a specified AASHTO T-99 density of 90%.
2. Use reasonable care in handling and installation.
* Minimum covers are for H20 and H25 loads. See Table 10.1 for construction load requirements. Minimum covers are measured from top of pipe to bottom of flexible pavement or top of pipe to top of rigid pavement. Minimum cover must be maintained in unpaved traffic areas.

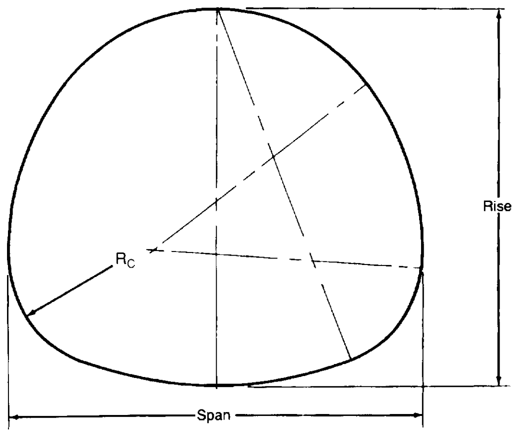


AISI-19

Height-of-Cover Limits for Structural Plate Horizontal Elliptical Pipe
E80 Live Load • 6 x 2 in. Corrugation

Pipe Size	Span ft-in.	Rise ft-in.	R_T in.	R_s in.	Minimum* Cover in.	Minimum Specified Thickness Required in.	Maximum Cover (ft) Over Pipe for Side and Haunch Soil Bearing Capacity of 3 tons/ft^2
24 E 15	7-4	5-6	54.00	26.50	24	0.111	24
27 E 15	8-1	5-9	60.88	26.50	24	0.111	21
30 E 15	8-10	6-0	67.75	26.50	24	0.140	19
30 E 18	9-2	6-9	67.75	32.00	24	0.140	24
33 E 15	9-7	6-4	74.63	26.50	24	0.140	17
33 E 18	9-11	7-0	74.63	32.00	30	0.140	21
36 E 15	10-4	6-7	81.51	26.50	30	0.140	15
36 E 18	10-8	7-3	81.51	32.00	30	0.140	20
36 E 21	11-0	8-0	81.51	37.50	30	0.140	23
39 E 18	11-4	7-6	88.38	32.00	30	0.140	18
39 E 21	11-8	8-3	88.38	37.50	30	0.140	22
39 E 24	12-0	8-11	88.38	43.00	30	0.140	25
42 E 18	12-1	7-10	95.26	32.00	30	0.140	16
42 E 21	12-5	8-6	95.26	37.50	30	0.140	20
42 E 24	12-9	9-2	95.26	43.00	36	0.140	23
45 E 18	12-10	8-1	102.13	32.00	36	0.170	15
45 E 21	13-2	8-9	102.13	37.50	36	0.170	19
45 E 24	13-6	9-6	102.13	43.00	36	0.170	22
48 E 18	13-7	8-4	109.01	32.00	36	0.170	13
48 E 21	13-11	9-0	109.01	37.50	36	0.170	17
48 E 24	14-3	9-9	109.01	43.00	36	0.170	20
48 E 27	14-7	10-5	109.01	48.50	36	0.170	23
48 E 30	14-11	11-2	109.01	54.00	42	0.170	26

Notes:
1. Soil bearing capacity refers to the soil in the region of the pipe haunches. See Chapter 10 for design of pipe envelope at pipe haunches. The remaining backtill around the ellipse must be compacted to a specified AASHTO T-99 density of 90%.
2. Use reasonable care in handlgng and installation.
* From top of pipe to bottom of tie.



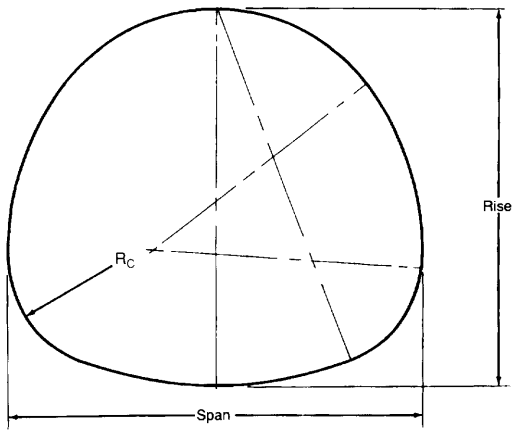
AISI-20

Height-of-Cover Limits for Structural Plate Underpass
H20 or H25 Live Load • 6 x 2 in. Corrugation

Size		R_c Corner Radius in.	Minimum Specified Thickness Required in.	Minimum* Cover in.	Maximum Cover (ft) Over Underpass for Soil Corner Bearing Capacity of 2 tons/ft^2
Span ft-in.	Rise ft-in.				
5-8	5-9	18	0.111	12	26
5-8	6-6	18	0.111	12	24
5-9	7-4	18	0.111	12	24
5-10	7-8	18	0.111	12	24
5-10	8-2	18	0.111	12	24
12-2	11-0	38	0.111	24	22
12-11	11-2	38	0.111	24	20
13-2	11-10	38	0.111	24	20
13-10	12-2	38	0.111	24	19
14-1	12-10	38	0.111	24	I9
14-6	13-5	38	0.111	24	19
14-10	14-0	38	0.111	24	19
15-6	14-4	38	0.111	24	15
15-8	15-0	38	0.111	24	15
16-4	15-5	38	0.140	36	15
16-5	16-0	38	0.140	36	14
16-9	16-3	38	0.140	36	14
17-3	17-0	47	0.140	36	17
18-4	16-11	47	0.170	36	16
19-1	17-2	47	0.170	36	15
19-6	17-7	47	0.170	36	15
20-4	17-9	47	0.188	36	14

Notes:
1. Soil bearing capacity refers to the soil in the region of the pipe corners. See Chapter 10 for design of pipe envelope at pipe corners. The remaining backfill around the underpass must be compacted to a specified AASHTO T-99 density of 90%.
2. Use reasonable care in handling and installation.

* Minimum covers are for H20 and H25 loads. See Table 10.1 for heavy construction load requirements. Minimum covers are measured from top of pipe to bottom of flexible pavement or top of pipe to top of rigid pavement. Minimum cover must be maintained in unpaved traffic areas.

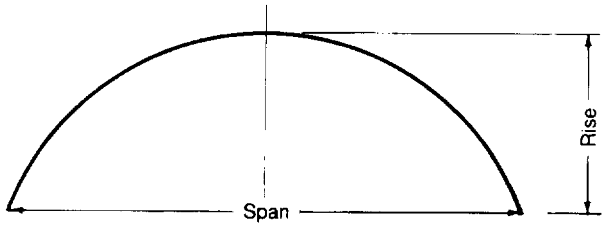


AISI-21

Height-of-Cover Limits for Structural Plate Underpass
E80 Live Load • 6 x 2 in. Corrugation

Size		R_C Corner Radius in.	Minimum Specified Thickness Required in.	Minimum* Cover in.	Maximum Cover (ft) Over Underpass for Soil Corner Bearing Capacity of 2 tons/ft^2
Span ft-in.	Rise ft-in.				
5-8	5-9	18	0.111	24	26
5-8	6-6	18	0.111	24	24
5-9	7-4	18	0.111	24	24
5-10	7-8	18	0.111	24	24
5-10	8-2	18	0.111	24	24
12-2	11-0	38	0.140	36	22
12-11	11-2	38	0.140	36	20
13-2	11-10	38	0.140	36	20
13-10	12-2	38	0.140	36	17
14-1	12-10	38	0.140	36	17
14-6	13-5	38	0.140	36	19
14-10	14-0	38	0.140	36	19
15-6	14-4	38	0.140	48	12
15-8	15-0	38	0.140	48	13
16-4	15-5	38	0.140	48	13
16-5	16-0	38	0.140	48	11
16-9	16-3	38	0.140	48	11
17-3	17-0	47	0.140	48	15
18-4	16-11	47	0.170	48	14
19-1	17-2	47	0.170	48	13
19-6	17-7	47	0.170	48	13
20-4	17-9	47	0.188	48	12

Notes:
1. Soil bearing capacity refers to the soil in the region of the pipe corners. See Chapter 10 for design of pipe envelope at pipe corners. The remaining backfill around the underpass must be compacted to a specified AASHTO T-99 density of 90%.
2. Use reasonable care in handling and installation.
* From top of pipe to bottom of tie.



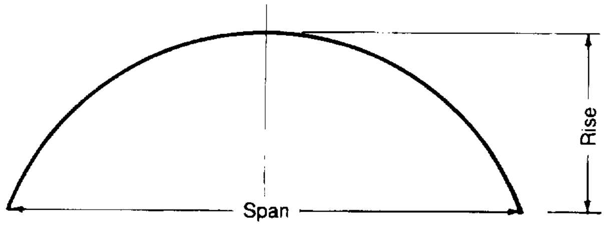
AISI-22

Height-of-Cover Limits for Structural Plate Arches
H20 or H25 Live Load • 6 x 2 in. Corrugation

$\frac{\text{Rise}}{\text{Span}} \geq 0.30$

Span, ft	Min.* Cover, in.	Maximum Cover (ft) for Specified Thickness (in.)						
		0.111	0.140	0.170	0.188	0.218	0.249	0.280
5	12	81	120	157	176	205	234	264
6	12	68	101	131	146	171	195	220
7	12	58	86	112	125	146	168	188
8	12	51	75	98	111	128	146	165
9	24	45	67	87	97	114	130	146
10	24	40	60	78	87	102	117	132
11	24	37	54	71	79	93	106	120
12	24	34	50	65	73	85	97	110
13	24	31	46	60	67	79	90	101
14	24	29	43	56	62	73	83	94
15	24	27	40	52	58	68	78	88
16	24	25	37	49	54	64	73	82
17	36	24	35	45	51	60	68	77
18	36	23	33	42	47	55	63	71
19	36	18	31	38	43	50	58	65
20	36		28	35	40	47	53	60
21	36		27	32	36	43	49	56
22	36		21	31	33	39	45	51
23	36			27	31	36	41	46
24	36			21	28	33	38	43
25	48				22	31	35	39
26	48					24	32	35

Notes:
1. Arches with R/S less than 0.30 require special design.
* Minimum covers are for H20 and H25 loads. See Table 10.1 for heavy construction load requirements. Minimum covers are measured from top of pipe to bottom of flexible pavement or top of pipe to top of rigid pavement. Minimum cover must be maintained in unpaved traffic areas.



AISI-23

Height-of-Cover Limits for Structural Plate Arches
E80 Live Load • 6 x 2 in. Corrugation

$\frac{\text{Rise}}{\text{Span}} \geq 0.30$

Span, ft	Min.* Cover, in.	Maximum Cover (ft) for Specified Thickness (in.)						
		0.111	0.140	0.170	0.188	0.218	0.249	0.280
5	24	81	120	157	176	205	234	264
6	24	68	101	131	146	171	195	220
7	24	58	86	112	125	146	168	188
8	24	51	75	98	111	128	146	165
9	24	45	67	87	97	114	130	146
10	24	40	60	78	87	102	117	132
11	30	37	54	71	79	93	106	120
12	30	34**	50	65	73	85	97	110
13	36	31 **	46	60	67	79	90	101
14	36	29**	43	56	62	73	83	94
15	36	24**	40	52	58	68	78	88
16	48	23**	37	49	54	64	73	82
17	48	16**	35	45	51	60	68	77
18	48	14**	35	42	47	55	63	71
19	48	13**	31	37	43	50	58	65
20	48		28	33	40	47	53	60
21	60		20	31	35	43	49	56
22	60		16	27	31	39	45	51
23	60			21	28	35	41	46
24	60			17	22	31	37	43
25	60				19	24	33	39
26	60					21	24	35

Notes:
1. Arches with R/S less than 0.30 require special design.
* From top of pipe to bottom of tie.
** These structural plate arches require additional minimum cover.

AISI-24

Minimum Cover In Feet for Airplane Wheel Loads on Flexible Pavements* - 2-2/3 x 1/2 in. Corrugation

Specified Thickness in.	Pipe Diameter, in.								
	12	18	24	36	48	60	72	84	96
Case 1. Loads to 40,000 Lb. - Dual Wheels									
.064	1.0	1.0	1.0	1.5	2.0				
.079	1.0	1.0	1.0	1.5	2.0				
.109			1.0	1.0	1.5	2.0			
.138				1.0	1.5	1.5	2.0		
.168				1.0	1.0	1.5	1.5	2.0	2.0
Case 2. Loads to 110,000 Lb—Dual Wheels									
.064	1.5	1.5	1.5	2.0	2.5				
.079	1.5	1.5	1.5	2.0	2.5				
.109			1.5	1.5	2.0	2.5			
.138				1.5	2.0	2.0	2.5		
.168				1.5	1.5	2.0	2.5	2.5	2.5
Case 3. Loads to 750 000 Lb—Dual-Dual									
.064	2.0	2.0	2.0	2.5	3.0				
.079	2.0	2.0	2.0	2.0	2.5				
.109			2.0	2.0	2.5	2.5			
.138				2.0	2.0	2.5	3.0		
.168				2.0	2.0	2.0	2.5	3.0	3.0
Case 4. Loads to 1.5 Million Lb									
.064	2.5	2.5	2.5	2.5	3.0				
.079	2.5	2.5	2.5	2.5	2.5				
.109			2.5	12.5	2.5	2.5			
.168				2.5	2.5	2.5	2.5	3.0	3.0
Diam.	12	18	24	36	48	60	72	84	96

Notes:
1. See Table AISI-5 (E 80 requirements) for maximum cover.
2. Backfill around pipe must be compacted to a specified AASHTO T-99 densitiy of 90%.
3. Use reasonable care in handling and installation.
4. Minimum cover is from top surface of flexible pavement to top of CSP.
5. Loads are total load of airplane.
6. Seam strength must be checked for riveted pipe.

* From "Airport Drainage," U.S. Dept. of Transportation, F.A.A., 1994.

AISI-25

Minimum Cover In Feet for Airplane
Wheel Loads on Flexible Pavements* - 5 x 1 in. and 3 x 1 in. Corrugations

Specified Thickness in.	Pipe Diameter, in.							
	36	48	60	72	84	96	108	120
Case 1. Loads to 40,000 Lb.—Dual Wheels								
.064	1.0	1.5	1.5	2.0	2.0	2.5		
.079	1.5	1.5	1.5	2.0	2.0	2.5		
.109	1.0	1.0	1.5	1.5	1.5	2.0	2.0	2.0
.138	1.0	1.0	1.0	1.5	1.5	1.5	2.0	2.0
.168	1.0	1.0	1.0	1.5	1.5	1.5	1.5	2.0
Case 2. Loads to 110,000 Lb—Dual Wheels								
.064	1.5	2.0	2.0	2.5	2.5	3.0		
.079	1.5	1.5	2.0	2.5	2.5	2.5	3.0	
.109	1.5	1.5	2.0	2.0	2.5	2.5	2.5	3.0
.138	1.5	1.5	1.5	2.0	2.0	2.5	2.5	2.5
.168	1.5	1.5	1.5	1.5	2.0	2.0	2.5	2.5
Case 3. Loads to 750,000 Lb—Dual-Dual								
.064	2.0	2.0	2.5	2.5	3.0	3.5		
.079	2.0	2.0	2.5	2.5	3.0	3.0	3.5	
.109	2.0	2.0	2.0	2.5	2.5	3.0	3.0	3.0
.138	2.0	2.0	2.0	2.0	2.5	2.5	2.5	3.0
.168	2.0	2.0	2.0	2.0	2.0	2.5	2.5	2.5
Case 4. Loads to 1.5 Million Lb								
.064	2.5	2.5	2.5	3.0	3.0	3.5		
.079	2.5	2.5	2.5	2.5	3.0	3.0	3 5	
.109	2.5	2.5	2.5	2.5	2.5	3.0	3.0	3.5
.138	2.5	2.5	2.5	2.5	2.5	2.5	3.0	3.0
.168	2.5	2.5	2.5	2.5	2 5	2 5	2 5	3 0
Diam.	36	48	60	72	84	96	108	120

Notes:
1. See Table AISI-5 (E80 requirements) for maximum cover.
2. Backfill around pipe must be compacted to a specified AASHTO T-99 densitiy of 90%.
3. Use reasonable care in handling and installation.
4. Minimum cover is from top surface of flexible pavement to top of CSP.
5. Loads are total load of airplane.
6. Seam strength must be checked for riveted pipe.

* From "Airport Drainage," U.S. Dept. of Transportation, F.A.A., 1994.

AISI-26				
Minimum Cover in Feet for Airplane Wheel Loads on Rigid Pavements—* (All Corrugations)				
Pipe Diameter, in.	**15,000 lb. Single Wheel**	**25,000 lb. Single Wheel**	**100,000 lb. Twin Assembly**	**265,000 lb. Twin-Twin Assembly**
6-60	0.5	0.5	11.0	1.0
66-108	1.0	1.0	1.5	1.5

Notes:
1. See Table AISI-5,-6,-8 for maximum cover.
2. Backfill around pipe must be compacted to a specified AASHTO T-99 density of 90%.
3. Use reasonable care in handling and installation.
4. Minimum cover is from bottom of slab to top of pipe.
5. Loads are not total loads but loads per wheel or assembly.
6. Minimum cover for C5A airplane is same as 100,000 lb. assembly.

* From "Development of Minimum Pipe-Cover Requirements for C-5A and Other Aircraft Loadings," C.C. Calhoun, Jr. and H.H. Ulery, Jr., U.S. Army WES, Vicksburg, MS, Paper S-73-65, November 1973.

AISI-27				
Minimum Cover in Feet for Airplane Wheel Loads on Flexible Pavements*— 6 x 2 in. Corrugation				
DualWheels With Loads To	**40,000 lb.**	**110,000 lb.**	**750,000 lb.**	**1.5 Million lb.**
Minimum Cover	D/8 but not less than 1.0 feet	D/6 but not less than 1.5 feet	D/5 but not less than 2.0 feet	D/4 but not less than 2.5 feet

Notes:
1. See Table AISI-8 for maximum depth of cover.
2. Backfill around pipe must be compacted to a specified AASHTO T-99 density of 90%.
3. Use reasonable care in handling and installation.
4. Minimum cover is from top surface of flexible pavement to top of CSP.
5. Loads are total load of airplane.

* From "Airport Drainage," U.S. Dept. of Transortation, F.A.A., 1994.

STRUCTURAL DESIGN OF STANDARD STRUCTURES BY THE LRFD METHOD

Load and Resistance Factor Design (LRFD) is a method of proportioning structural elements (the pipe) by applying factors to both the loads (load factors) and the nominal strength levels (resistance factors). The specified factors are based on the mathematical theory of reliability and a statistical knowledge of load and material characteristics. The load factors are multipliers (typically greater than 1.0) that take account of the variability of different types of loads, such as earth loads and live loads. Thus, the pipe must be designed to resist a combination of factored earth loads and factored live loads plus impact.

Resistance factors are tytpically 1.0 or lower. They account for the possible reduction in the strength of the structural materials involved. While LRFD designs don't openly display the degree of safety (the factor of safety) as such, it is essentially the ratio of the factored load divided by the factored resistance.

LRFD methods may be found in both the AASHTO *LRFD Bridge Design Specifications* and in ASTM Standard Practice A796/A796M. AASHTO has set a goal to use the LRFD method for all new construction. ASTM A976/A796M includes both Allowable Stress Design (ASD) and LRFD as alternative procedures. ASTM LRFD is a simplified version of AASHTO LRFD, which involves additional factors and alternative live loads. The referenced documents should be referred to for complete details.

DESIGN OF OTHER STRUCTURES

The design methods discussed previously in this chapter address standard corrugated steel pipe and plate structures. They are based on the American Iron and Steel Institute (AISI) working stress design method, or similar AASHTO methods, which have been used successfully for traditional products for over 60 years. This includes round pipe and arches with a maximum span of 26 feet as well as pipe arches and underpasses with a maximum span of 21 feet. However such methods are not applicable to structures with long spans or high bending moments, such as box culverts and long span shapes and deep corrugated structures.
Because of their size or shape, the design of those structures is based on simplified methods derived from finite element evaluations, or direct finite element designs using software such as CANDE. The most recent design methods are included in the AASHTO *LRFD Bridge Design Specifications* and in the *Canadian Highway Bridge Design Code* (CHBDC).

A limited discussion of product background and design aspects for these larger or special shape structures follows. However, reference should be made to the above references for further design information for those structures.

Long Span Structures

Structural design of long span structures with the 6 inch x 2 inch corrugation follows the traditional ring compression method with additional checks to account for size and flexibility. Designs can be made with both the AASHTO *LRFD Bridge Design Specifications* and the AASHTO *Standard Specifications for Highway Bridges*. An empirical table of minimum thickness is specified based on a top radius from 15 to 25 feet. Long span structures, like box culverts, are limited to backfill materials that meet AASHTO M 145 requirements for A1, A2-4, A2-5, and A3 materials. These materials must be compacted to a minimum 90% modified Proctor density (90% AASHTO T-180). Long span structures are installed in accordance with the AASHTO *LRFD Bridge Construction Specifications*, Section 26.

Corrugated Steel Box Culvert Development

As illustrated in Figure 7.17, corrugated steel box culverts have a low, wide rectangular profile that necessitates the use of special design methods. Because of their nearly flat crowns and large span/rise ratios, box culverts behave differently than traditional soil-metal structures and must be designed in a different way. The first corrugated metal box culverts were built in 1975 using an empirical design method. Within a few years, a considerable number had been constructed and the demand for larger sizes increased to a point where completely empirical design procedures were no longer appropriate.

A study was initiated at the University of California - Berkeley (Duncan et al) to develop a rational design method for aluminum box culverts. The first phase of the study was a series of finite element analyses to evaluate the bending moments and the axial forces in box structures under loads imposed by backfill and live loads. In the next phase, full scale tests were conducted on instrumented box structures to provide a basis for calibrating the finite element analysis with measured behavior. This was augmented by several state DOTs that conducted field live load tests on each box culvert installed.

In 1987, AASHTO adopted a simplified design method for corrugated steel and aluminum box culverts based on the box culvert geometry limits represented in the various studies. This method is limited to box structures with spans through 25 feet 5 inches and rises through 10 feet 6 inches. Cover limits range from a minimum of 1.4 feet to a maximum of 5 feet.

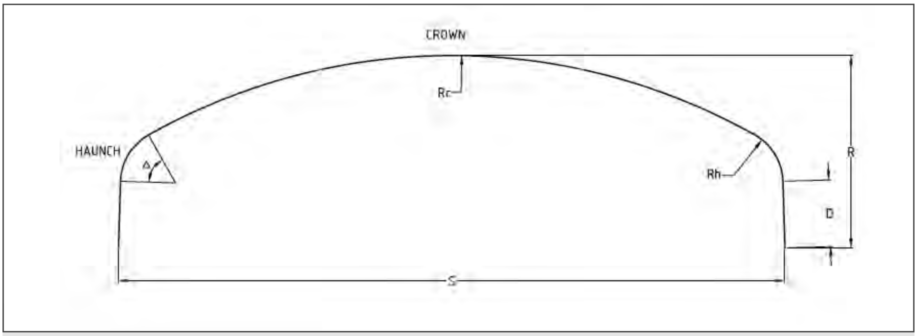


Figure 7.17 The Standard Corrugated Steel Box Culvert shape

Structural Plate Box Culverts

Many corrugated steel box culverts are made with 6 inch x 2 inch corrugated steel structural plate, strengthened with longitudinally spaced steel ribs to provide the necessary moment resistance. Design became standardized with the advent of the AASHTO box culvert design method and ASTM specification A 964/A 964M.

Steel box culverts are not ring compression structures. Rather, they act as soil supported, semi-rigid frames and are designed on the basis of bending moments and plastic moment strength. A design method is available in Section 12 of both the AASHTO *LRFD Bridge Design Specifications* and the AASHTO *Standard Specifications for Highway Bridges*. The following table lists the geometry limits that are applicable to the AASHTO LRFD method. Box structures outside the geometry limits in Table 7.9 must be designed using more rigorous, finite element methods.

Table 7.9

Standard Corrugated Steel Box Culvert Geometry Limits

Dimension	Minimum	Maximum
Span	8 ft - 0 in.	25 ft - 5 in.
Rise	2 ft - 6 in.	10 ft - 6 in.
Crown Radius	–	24 ft - 9 1/2 in.
Haunch Radius	2 ft - 6 in.	–
Included Angle of Haunch	50°∞	70°∞
Leg Length (to bottom of plate)	0 ft - 4 3/4 in.	5 ft - 11 in.

Standard corrugated steel box culverts are installed in accordance with the AASHTO *LRFD Bridge Construction Specifications*, Section 26. They require backfill materials classified by AASHTO M 145 as A1, A2-4, A2-5, and A3, compacted to a minimum 95% standard Proctor density (AASHTO T-99).

Deep Corrugated Steel Box Culverts

The Canadian Highway Bridge Design Code (CHBDC) box culvert design method was developed from the 1993 AASHTO design method. With the introduction of deep corrugated plate it became practical to increase the span of box culverts beyond the limits of the original 1984 Duncan study. Deep corrugated steel box culverts manufactured with 15 x 5.5 inch and 16 x 6 inch corrugation profiles have reached spans of over 50 feet. The design of structures with such long spans is complex. Performance is related to the interaction of the structure and the surrounding soil and, thus, the properties of the surrounding soil have a major effect on performance. For spans greater than 26 feet and/or rises greater than 10 feet 5 inches, the forces in the structure are calculated by rigorous methods of analysis, taking into account the beneficial effects of soil-structure interaction. All deep corrugated box structures can be analyzed with finite element programs.

Other Deep Corrugated Structures (Arches, Ellipses, and Round)

A limit states design method that reflects the variability in both loads and resistance of structural elements is used for deep corrugated structures. It is calibrated to provide a more uniform and quantifiable level of reliability than can be achieved with working stress design (WSD). The *Canadian Highway Bridge Design Code* (CHBDC) introduced this design method for deep corrugated structures in 2001 and updated it in 2006. The

CHBDC design code has been adopted by many countries around the world as the design method of choice for deep corrugated structures. The CHBDC method is based on limit states design philosophy (ultimate strength principles) rather than traditional working stress or service load design methods.

Deep corrugated steel structures using 15 x 5.5 inch and 16 x 6 inch corrugation profiles have reached spans of 80 feet. The design of structures with such long spans is complex. Performance is related to the interaction of the structure and the surrounding soil and, thus, the properties of the surrounding soil have a major effect on performance. The CHBDC design method quantifies the strength of both the soil and the structure.

The AASHTO design method was developed for 6 x 2 inch structural plate. It considers axial thrust effects only because the flexural rigidity of these plates is relatively small and bending moment can be ignored in most practical cases (when minimum cover levels are adequate). In contrast, deep corrugated plate has three times the bending strength and ten times the elastic stiffness of 6 x 2 inch plate. As a result, bending moments cannot be ignored in deep corrugated structures. A design procedure that includes the effects of bending moments, such as the CHBDC method, is necessary to account for the increased stiffness of deep corrugated plate. In addition to the CHBDC method, deep corrugated soil-metal structures can be analyzed by rigorous design methods, such as with finite element programs, taking into account the beneficial effects of soil-structure interaction.

The CHBDC design method involves the following steps:

1. Check minimum allowable cover
2. Calculate dead load thrust
3. Calculate live load thrust
4. Calculate earthquake thrust
5. Calculate the total factored thrust
6. Calculate the compressive stress
7. Calculate the wall strength in compression
8. Check combined bending and axial strength during construction
9. Check combined bending and axial strength in the ultimate limit state
10. Check seam strength
11. Check difference in plate thickness of adjacent plates
12. Calculate footing loads
13. Check plate radius of curvature

OTHER DESIGN REQUIREMENTS

End Treatment

Designing the ends of a flexible culvert requires additional considerations beyond those addressed in the ring compression design of the culvert barrel. End treatment design must also consider any unbalanced soil loadings due to skews or excessive cross slopes, the residual strength of any skew cut or bevel cut ends employed, as well as possible hydraulic action due to flow forces, uplift, and scour.

Pipe skewed to an embankment (pipe that cross through at an angle) are subjected to unbalanced soil loads through and beyond the area of the fill slope. The unbalance is easily seen by cutting a section across the pipe perpendicular to its longitudinal axis. The amount of unbalance depends on the degree of skew (angle), the diameter (span) of the pipe, and the slope of the embankment. Unbalanced soil loads typically are not a serious consideration when skews are maintained within the limits of Figure 7.18. Where multiple runs of pipe are used, the total span of the entire run, including the space between the pipes, must be considered in lieu of the span or diameter of a single pipe.

Where skews must exceed these limits, the embankment may be shaped or warped to balance the loads and ensure side support. Figure 7.20 provides typical examples of both properly and improperly balanced end treatments. Alternatively, full headwalls can be used. A rigid headwall, designed to carry the thrust forces of the cut end of the pipe can provide for nearly any degree of skew required.

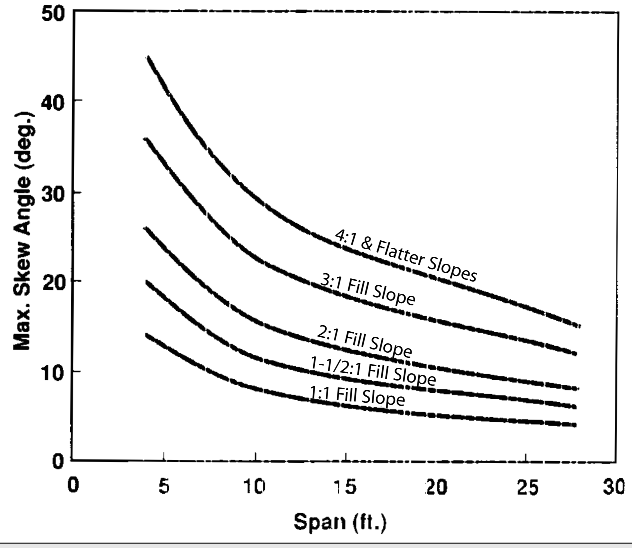
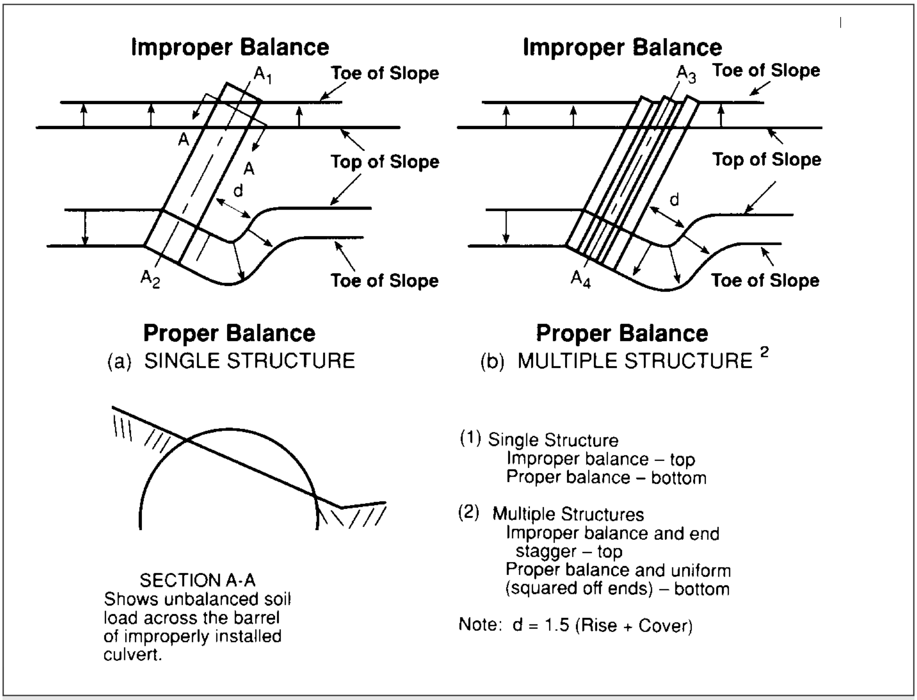


Figure 7.18 Suggested limits for skews to embankments unless the embankment is warped for support or full head walls are provided.

For most applications square end pipe is recommended. In multiple runs, the ends must be extended so they are aligned perpendicularly as shown for "Proper Balance" in Figure 7.20 (b). Adequate side support at the ends of multiple runs cannot be achieved if they are staggered as shown for "Improper Balance" in Figure 7.20 (b).

■ **Figure 7.19** Long span grade separation.



■ **Figure 7.20** Properly and improperly balanced (warped) embankment fills for single and multiple culvert installations.

Skew cut, bevel cut or skewed/bevel cut ends are sometimes used for hydraulic or aesthetic reasons. When the pipe ends are cut in any fashion, the compression ring is interrupted and pipe strength in the cut area is limited to the bending strength of the corrugation. Simple skew cut ends can generally handle soil and installation loads if they are limited to the skew angle limits of Figure 7.18. However, hydraulic flow forces must be considered separately. Headwalls, concrete collars, and other reinforcements can be provided as necessary.

Bevel cuts, as shown in Figure 7.21, can be done in several fashions. Step bevels are recommended for all pipe sizes. Step bevels are typically limited to 3:1, 2:1 or steeper slopes on long span and larger structural plate pipe, depending on their rise (height). Full and partial bevels are typically applicable only to smaller pipe as suggested by Table 7.10.

Full bevels are not recommended for multiple radius shapes such as pipe arch and underpass or with bevel slopes flatter than 3:1. Even then, pipe with full inverts must have the invert trimmed, as shown for a partial bevel.

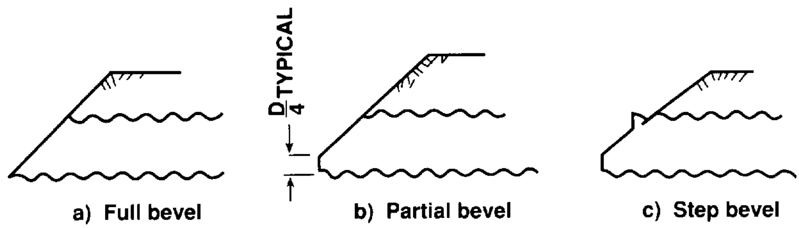


Figure 7.21 Types of beveled ends.

Table 7.10

Recommended Diameter (or Span) Limits (in.) for Full and Partial Bevel Cut Ends (Slope Collars, Toe Anchorage, etc. are required)

Specified Thickness in.	Corrugation Type: 2-2/3 x 1/2 in. 3/4 X 3/4 X 7-1/2 in. 3/4 X 1 X 11-1/2 in.	3 x 1 in. 5 x 1 in.	6 x 2 in.
.064	48	78	
.079	54	84	
.109/.111	60	96	156
.138/.140	66	108	168
.168 /.170 /.188	72	114	180
.218			198
.249			210
.280			216

All types of bevel cut ends typically require protection, especially when hydraulic flow forces are anticipated. The cut portion should be anchored to slope pavement, slope collars or headwalls at approximately 18 inch intervals. Cutoff walls or other types of toe anchorage are recommended to avoid scour or hydraulic uplift problems.

Skew bevel cut ends may be used where they meet the criteria for both skew and bevel cut ends.

Hydraulic forces on inlet or outlet ends are difficult to quantify. When structures are designed to flow full under pressure, where flow velocities are high or where flows are expected to increase abruptly, significant hydraulic forces should be anticipated. Alternatively, equalizer pipe, slow flowing canal crossings, etc., generally do not provide the same level of concern.

Where significant hydraulic action is anticipated, support and protection of the pipe end (especially the inlet), erosion of the embankment fill, undercutting or piping of the backfill or bedding, and hydraulic uplift, become important design considerations. Slope collars, or slope pavements with proper pipe end anchorage can provide support for the pipe end and reduce erosion concerns. A compacted 1 foot thick clay cap over the fill slope, with proper erosion protection such as riprap, helps keep water from the backfill. Toe or cutoff walls, placed to an adequate depth, keep flow from undermining the invert and provides anchorage for the pipe end.

Half headwalls with cutoff walls (especially on the inlet end), as well as more elaborate full headwalls, not only stiffen the pipe end against damage from water energy, but also improve the efficiency of the inlet. Figures 7.22 – 7.25 show typical headwall treatments.

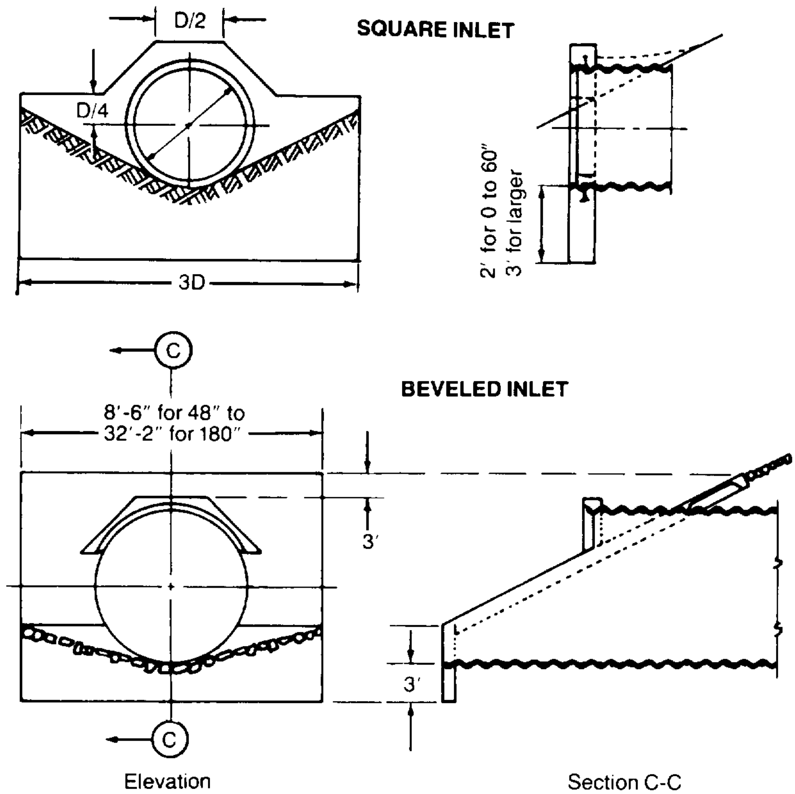
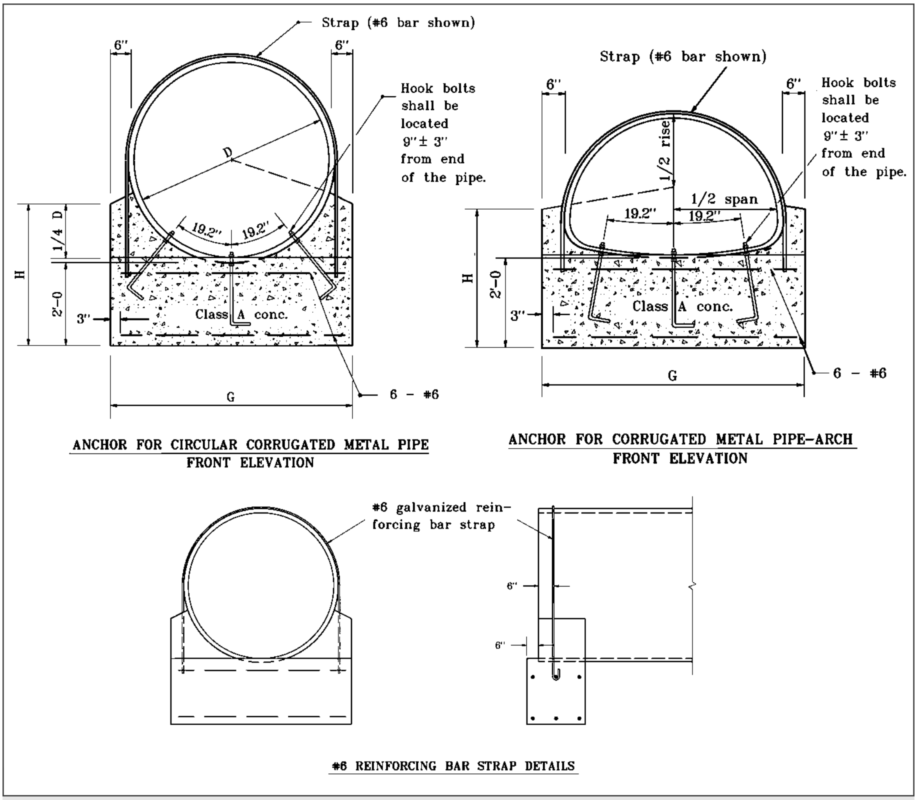
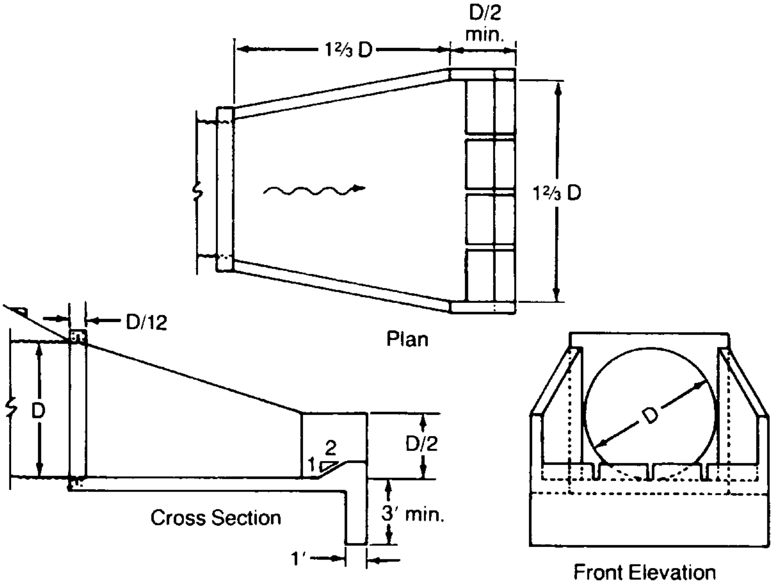


Figure 7.22 Treatment of inlet end of large corrugated steel structures as recommended by the Federal Highway Administration.



■ **Figure 7.23** Treatment of inlet end of corrugated steel pipes or structural plate pipes as recommended by the State of Indiana, Department of Transportation.

■ **Figure 7.24** Pipe arch can be installed in limited headroom situations with shallow cover.



■ **Figure 7.25** Treatment of outlet end of large corrugated steel structures.

Besides improving hydraulic flow and supporting any skew or bevel cut ends, these treatments provide cutoff walls below and beside the pipe to protect the backfill and embankment slope from piping and erosion. By decreasing the quantity of seepage from the upstream water course into the granular backfill, they reduce the hydraulic uplift (pore pressure) forces on the pipe.

Most highway and railway design offices have adequate design standards suitable to their terrain. Reference to these is valuable for design of headwalls, riprap protection and slope pavements.

Camber

An embankment exerts more load on the foundation at the center of the embankment than at the toe of the slope, so more settlement will occur in the center area. A corresponding settlement of the conduit will occur. Hence, the foundation profile should be cambered longitudinally as illustrated in Figure 7.26. The upstream half of the pipe may be laid on almost a flat grade and the downstream half on a steeper grade. The mid-ordinate of the curve should be determined by the soils engineer. For further details on foundation preparation, see Chapter 10 Installation.

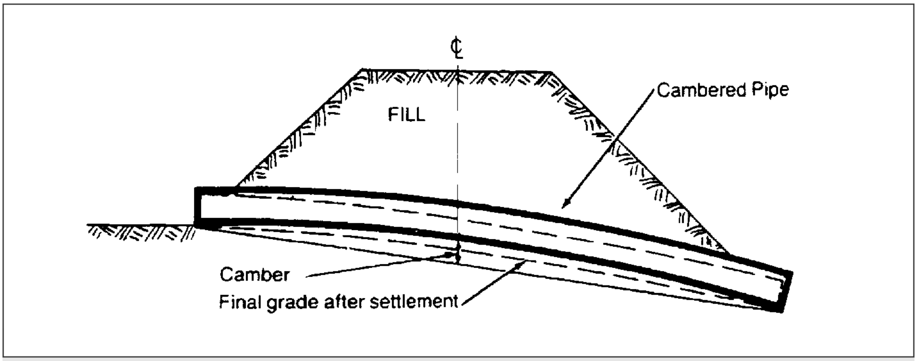


Figure 7.26 Camber allows settlement of a culvert under a high fill. Most of the fall is in the outlet half. Diameters 10ft and smaller are easier to camber, as are the lighter wall thickness.

Temporary Bracing

During the construction of headwalls, the ends of structures may require temporary bracing to prevent distortion. The end of a conduit cut on an extreme skew and bevel typically requires support from shoring or bracing until the slope pavement is completed. However, it is not normal, necessary or recommended to brace steel pipe on a routine basis in an attempt to control shape change or deflection during construction. The desired results are best obtained by proper compaction of a suitable backfill material.

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CSP form for wind turbine generator footing.

CHAPTER eight

INTRODUCTION

Corrugated steel pipe has many uses other than for culvert and storm sewer applications. However, even these conventional applications have a myriad of fittings, steel manholes, etc. that in larger sizes and deeper cover applications need special design and reinforcement.

Nonstandard applications include using corrugated steel pipe as vertical shaft liners, standpipes, grouted in place reline (rehabilitation) structures, above ground aerial spans, and structural columns to name a few. When the pipe is not backfilled or if it is stood on end, structural considerations change. Standpipes, grouted in place pipes, and other applications, have hydrostatic buckling and floatation issues that must be recognized in both design and construction.

FITTINGS REINFORCEMENT

Standard and special fittings can be shop fabricated from corrugated steel pipe. Like the rest of a buried pipeline, the thrust in the fitting depends on its diameter and the loads acting upon it. As the fittings get larger or become more deeply buried, they reach a point where additional steel structural reinforcing members or tensile strips, must be used to reinforce the area where the mainline has been cut away to allow the branch hub to join it.

■ CSP reinforced fitting.

Table 8.1 provides a list of the largest fittings that do not require longitudinal reinforcement while carrying up to 10 feet of cover with H20 and H25 wheel loads. This is a simple check to cover smaller diameter applications. Reinforcement, when necessary, should

Table 8.1

Maximum CSP branch diameters that do not require longitudinal reinforcement under up to 10 ft cover and H20/H25 wheel loads**

2 2/3" x 1/2" CORRUGATIONS															
Wall Thickness	0.064"			0.079"			0.109"								
Maximum Cover	10'	20'	30'	10'	20'	30'	10'	20'	30'						
Main Diameter															
48"	48	36	24	48	42	30	48	48	36						
60"					54*	36*	24*	42	36						
3" x 1" and 5" x 1" CORRUGATIONS															
Wall Thickness	0.064"			0.079"			0.109"			0.138"			0.168"		
Maximum Cover	10'	20'	30'	10'	20'	30'	10'	20'	30'	10'	20'	30'	10'	20'	30'
Main Diameter															
60″	42	24	18	54	30	24		42	30						
72"	36	24	18	48	24	18	54	36	24						
84"	30	18	18	42	24	18	54	30	24		42	30			
96"	30	18	12	36	24	18	48	30	18	54	36	24			
108"				36	18	12	42	24	18	42	30	24			
120"				30	18	12	42	24	18	48	30	24	54	36	24
132"							36	24	18	42	30	18	48	36	24
144"										42	24	18	48	30	24
3/4" x 3/4" x 7 1/2" SPIRAL RIB PIPE															
Wall Thickness	0.064"			0.079"			0.109"			0.138"					
Maximum Cover	10'	20'	26'	10'	20'	29'	10'	20'	30'	10'	20	' 30'			
Main Diameter															
48"	42	30	24	48	30	24	48	42	30						
60"	36	24		48	30	24	60	36	24						
72"				42	24		54	30	24						
84"							48	30	24	60	42	30			
96"							42	24	18	54	36	30			
108"										48	36	30			
3/4" X 1" x 11 1/2" SPIRAL RIB PIPE															
Wall Thickness	0.064"			0.079"			0.109"								
Maximum Cover	10'	20'	26'	10'	20'	29'	10'	20'	30'						
Main Diameter															
48"	42	30	24	48	30	24	48	42	30						
60"	36	24		42	30	24	60	36	24						
72"				36	24		48	24	24						
84"				36	24		42	24	18						
96"							42	24	18						
108"							36	24							

Notes:

* 60" 16 gage main diameter not available. Use 54" main diameter.

** Branch diameters listed assume 90 degree tee connections to the mainline. For wyes and other conditions, increase the branch diameter to d/sinØ before entering the table. Ø is the acute angle of the pipe's intersection. d/sinØ is equal to the span of the main cutout.

† Blank entries indicate cases not investigated. For intermediate branch diameters, or intermediate covers, interpolate or select the lower branch diameter. For branch angles other than 90 degrees (but no less than 30 degrees), use the span (major dimension of opening cut in main pipe for branch pipe) rather than the branch diameter.

be done in accordance with ASTM A 998. In some cases, circumferential reinforcement is also required. The ASTM standard not only provides tabulated solutions but also a complete design method. A computer program based on this method, CSPFIT, is available from the National Corrugated Steel Pipe Association.

There are several ways to reinforce most fittings and even more ways to attach the reinforcements. Most fabricators have their own detail depending on their tooling and inventory items. It is most economical to allow the fabricator to select the reinforcement means, while the specifier insists on reinforcement to ASTM A998 requirements.

Where steel structures are designed for storage (such as detention, retention and recharge systems) rather than flow, reinforcement can often be avoided by cutting a man-way or access door through the fitting. For example, rather than cutting out the full opening from the branch into to the mainline to fabricate a tee, a narrow doorway is cut just large enough to provide for adequate flow and personnel access through the tee.

Like a doorway, man-ways are typically cut two and a half to three feet wide and extend to the invert. The man-way is cut as tall as necessary (6' – 8" where possible) to provide easy access. The man-way does not require longitudinal reinforcement as long as its width along the axis of the mainline pipe does not exceed the diameter of the largest fitting (tap-in) in Table 8.1. The need for circumferential reinforcement should also be checked.

STEEL BULKHEADS

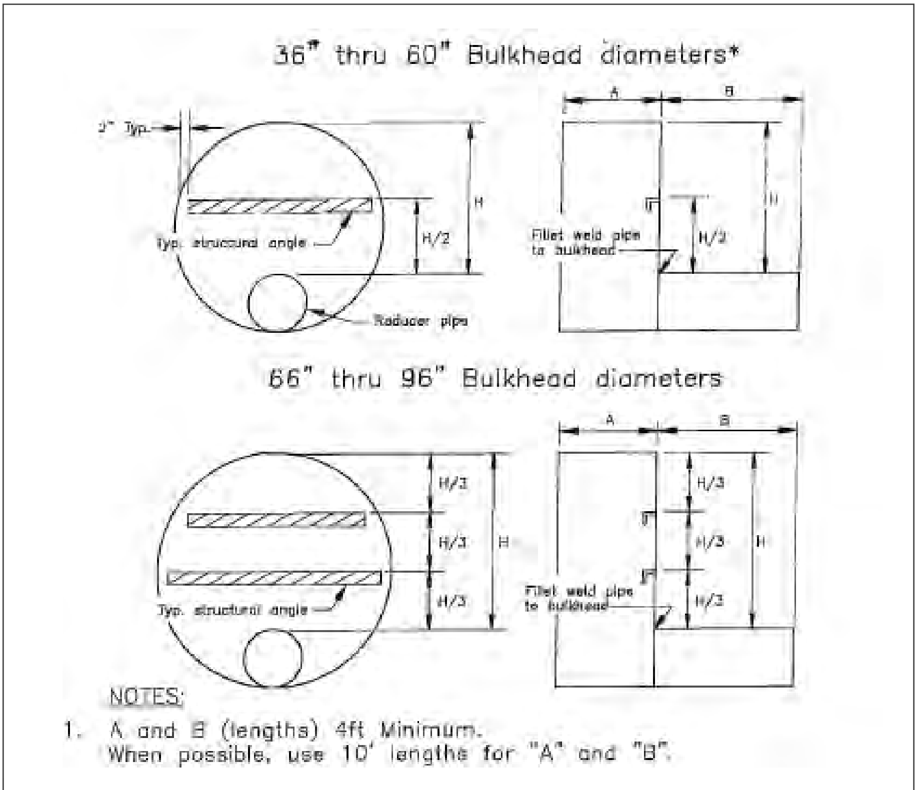
Steel bulkheads can be supplied with the pipe. They are widely used in detention and recharge systems and for transitions of smaller pipes into larger ones or as end plates. Typically they are continuously shop welded to CSP and either bolted or field welded to structural plate structures.

■ Shop attached CSP reinforced bulkheads.

The amount of reinforcement necessary varies with the diameter, depth of cover, and the thickness of the bulkhead plate itself. While it results in an overly conservative design, large diameter bulkheads can be handled traditionally by taking the reinforcements as a series of simple beams spanning the pipe end, with the bulkhead plate welded to them to develop composite action. In the opposite direction, the bulkhead plate itself can be analyzed as a continuous beam, spanning over the reinforcements.

A bulkhead welded to the pipe end more correctly acts as a fixed edge diaphragm. It may be designed using appropriate flat plate formulas from sources such as "Roark's Formulas For Stress and Strain", W.C. Young. Bending strength can be provided with a combination of the bulkhead plate thickness and steel reinforcements.

The basic equations to determine the necessary bending strength (required section modulus), reinforcement spacing and attachment welds, are provided below. The maximum reinforcement spacing depends on the bulkhead plate thickness. The spacing can be determined by taking the bulkhead plate as a second rectangular diaphragm with a width matching the spacing of the structurals. Welding requirements are provided that assure composite action of the plate and structural reinforcement.



■ **Figure 8.1** Bulkhead details.

Earth pressure $p = (h + 0.67\ D)g\ K_a / 144$

where:
P = design soil pressure on bulkhead, psi
h = height of cover, ft
D = diameter or rise of pipe, whichever is less, ft
g = soil density, typically taken as 120 pcf
K_a = active soil pressure coefficient (assume $K_a = 0.4$)

Bulkhead wall thickness $t_1 = [3w/(4\pi S_1)]^{1/2}$

where:
t_1 = required bulkhead wall thickness if an unreinforced, thick bulkhead plate were used, in.
$w = \pi\ (D/2)^2\ p$
$S_1 = F_y$ =36,000 psi (yield strength of steel reinforcement)

$$S_{req'd} = \frac{t_1^2}{6}$$

where: $S_{req'd}$ is the required section modulus of the composite, reinforced section used in lieu of a thick bulkhead plate (in^3 /ft of bulkhead width)

Max spacing, $b = [S_2 t^2/(\beta\ p]^{1/2}$

Where:
$S_2 = F_y$ = yield strength of the plate = 33,000 psi
β = diaphragm shape coefficient taken as 0.5
t = thickness of bulkhead plate chosen for use, in.

In fabricating these designs, the steel structural reinforcements must be located on the outside of the bulkhead. This insures that the flat plate will be in bending tension and will remain fully effective. To assure composite action, the bulkhead plate and reinforcement must be adequately welded together. Typically the reinforcements are welded to the plate with intermittent fillet welds sized to provide adequate shear flow between them. These attachment welds can be sized as follows.

Q = (A of one reinforcement) d

where: d is the distance between the neutral axis of the reinforcement and that of the completed composite section (reinforcement and plate)

$$q = VQ/I$$

where:

q = required weld strength of reinforcement, lbs/in.

V = maximum shear on the section (lbs/ft) which can be taken as 12 (span/2) p (in. lbs/ft)

I = moment of inertia of the welded, composite section (in^4) for one reinforced spacing

A specific weld strength and welding pattern can be selected by conventional means. However, limiting the maximum center to center weld spacing to 12 in., the following limits may be conservatively applied:

P_w = 700 L (lbs per weld) using an E70 electrode

where:

P_w = strength of a 1/16 in. fillet weld, lbs/in. 1/8 and 3/16 in. fillet welds respectively provide twice and three times this strength

L = length of each weld, in.

INTERNAL FLOW CONTROL STRUCTURES

It is sometimes necessary to incorporate flow control features within a drainage or detention system. In many cases, these flow controls are handled in the form of a weir plate that may have orifices or notches cut within it to limit the outflow at various elevations within the system. Corrugated steel pipes generally allow these features to be fabricated directly within the piping system, eliminating the need for transitioning to a junction chamber or other device to achieve the necessary flow controls.

In regards to the design methodology for internal weir plates is very similar to that described in the previous section for plate bulkheads. The primary difference is in the loads that need to be considered. Internal weir plates only need to be able to resist the hydrostatic loads of the water they are holding back within the pipe. Therefore, a conservative approach to determining the design pressure would be as follows:

Design pressure $p = (g \times H) / 144$

Where:

p = design pressure on the weir, psi

g = unit weight of water = 62.4 pcf

H = total height of weir plate, ft

■ Internal CSP reinforced weir.

Once this pressure is determined, it can be inserted into the same equation for determining the required plate thickness, required section modulus, allowable spacing for reinforcement, etc. as was given in the previous section for the design of bulkheads.

If a composite reinforced section is used, the reinforcement should be attached on the upstream side of the weir plate. Generally, the weir will be continuously welded to the pipe so water cannot leak through the seam.

CSP MANHOLES

Corrugated steel pipe makes an excellent, cost-effective manhole for use with CSP culverts, storm sewers and underground detention systems. The riser manhole and the shaft manhole are two common types that are used. These are shown below. The riser manhole is used where the mainline is a larger diameter than the manhole. The riser is typically aligned with the spring line of the main pipe rather than centered over the pipe. This not only transmits load more effectively, but also allows the ladder steps to transition smoothly to the floor.

■ CSP riser manholes in underground detention system.

CSP shaft-type riser.

Typically a shaft type manhole is set, open bottomed in a freshly poured concrete base slab. The slab extends out, beyond the manhole outside diameter (OD) far enough to keep the manhole from floating and is designed to be strong enough to handle any other vertical loads. Other vertical loads can include a wheel load applied to the manhole cover, the dead load or weight of the riser, and for shaft type manholes, any soil drag down load that is allowed to occur.

Either type of manhole is reinforced as any other fitting, in accordance with ASTM A998. However the shaft type manhole is less likely to need much reinforcement. In this case, the pipeline taps the manhole itself. The vertical shaft only sees the active soil pressure rather than the soil prism load exerted on a typical storm sewer of culvert.

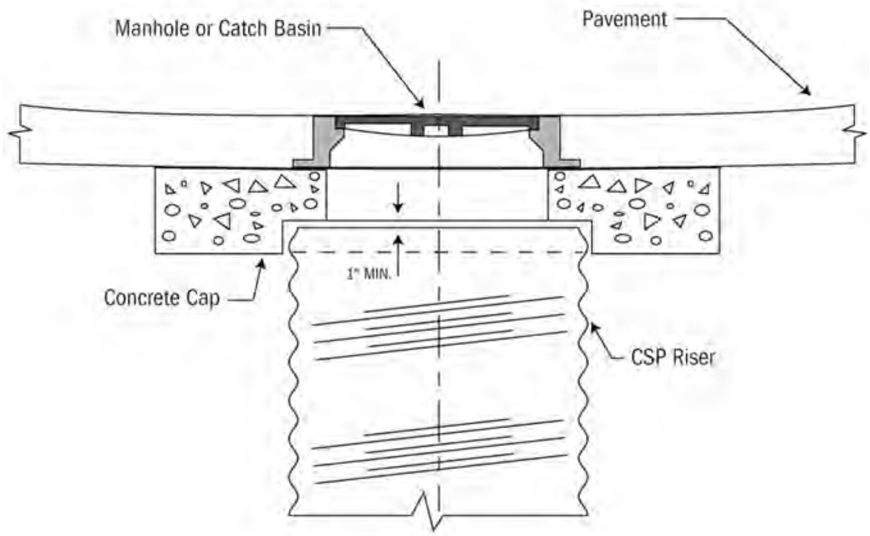
For riser type manholes, reinforcement of the connection to the mainline becomes an additional consideration. It is recommended that the manhole or catch basin inlet be supported at or near the surface by a concrete cap (actually a footing). The CSP riser is kept uncoupled from the cap so the cap floats, bearing down on the backfill and soil. This keeps the traffic load off the CSP manhole.

Drag-down loads are caused by soil settlement around the manhole riser. As the soil settles it attempts, through the friction force of the soil against the manhole, to drag the riser down with it. These loads can be very large. It is generally better to accommodate the movement (settlement) of the soil than try to design for loads of this magnitude.

Where necessary, the magnitude of the drag-down load can be estimated from the following equation:

$$Q = p_v \, \beta \, A_s$$

where: Q = dragdown force
p_v = average vertical soil pressure along height of riser, psf
β = 0. 20 to 0.025 for clay; 0.25 to 0.35 for silt; and 0.35 to 0.50 for sand.
A_s = surface area of the riser = DH
D = diameter, ft
H = height, ft



Notes:
1. A concrete cap shall be used on top of a riser when the riser is located within the area of vehicular traffic.
2. The concrete cap shall be sized and designed by others so that vehicular loads are transmitted to the soil and not to the riser.
3. The concrete cap shall be sized to provide an adequate bottom area based on the allowable bearing capacity of the soil.

Figure 8.2 Manhole cover detail.

Rather than attempt to design for these loads, it is often better to install a slip joint near the bottom, just above the mainline pipe(s). This is generally done by using a special band and wooden blocking devices as shown in Figure 8.3. Excessive loads split the blocks, which allows the riser to move down with the settlement and relieve the loads. Manholes taller than 10 feet or those backfilled with other than well compacted, granular materials, should have a slip joint located about 2 feet above the mainline pipe(s). With very tall risers, it is best to install at least one additional slip joint, farther up the riser. An estimate of the soil modulus for the contractor-placed backfill around the riser and the soil load on it, can be used to estimate the settlement that will occur. There needs to be enough potential slippage to accommodate expected settlements.

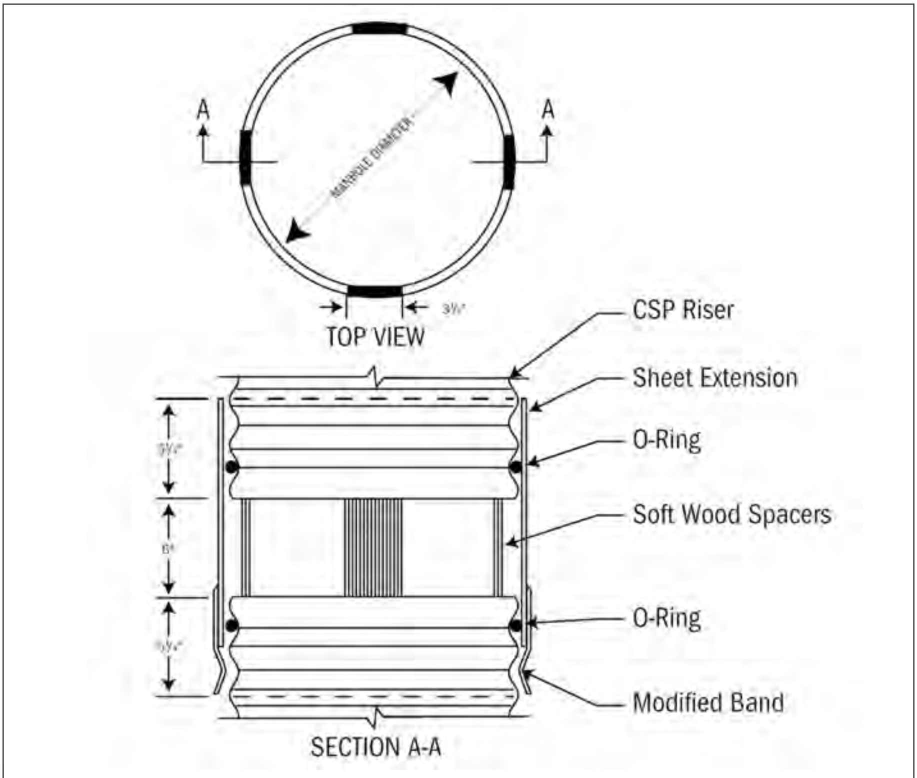


Figure 8.3 CSP riser slip joint.

Loads beyond these are primarily the horizontal pressures on the manhole. These include the active soil pressure, any ground water pressure, and lateral affects of live load pressures (near the surface). They are described as:

$$P_h = \gamma_w H_w + K_a\{\gamma H\ [1 - 0.33(H_w/H) + P_{LL}\}$$

where:

γ_w = density of water = 62.4 pcf
H_w = height of the ground water above the location being evaluate, ft
K_a = active soil pressure coefficient = $\tan^2(45\text{-}\Phi/2)$

where: Φ is the internal friction angle of soil (for clays, take Φ as 28 degrees)

γ = density of the soil of the moist soil, typically taken as 120 pcf
H = height of soil cover above the location being evaluated, ft
P_{LL} = live load pressure at the depth evaluated, psf

HYDROSTATIC BUCKLING

Conduits that are not buried in compacted soil while subjected to external hydrostatic pressure may be designed for buckling assuming they act as circular tubes under uniform, external pressure. No active or passive soil pressure is available for support in this condition and the pipe ring itself must resist instability, including the effects of bending moments resulting from out-of-roundness.

"Theory of Elastic Stability", Timoshenko and Gere, details methods of analysis for such thin tubes. However, no extensive correlation has been made between these buckling equations and corrugated pipe. Field experience and the few tests that have been done suggest that a modified form of the equations provides a conservative estimate of the collapse pressure of corrugated steel pipe.

The Timoshenko buckling equation is:

$$P_{cr} = \frac{3EI}{(1 - \mu^2) R^3}$$

Where: P_{cr} = critical pressure, psi
E = modulus of elasticity of pipe wall, psi
I = moment of inertia of pipe wall, in.^{4}/in.
μ = poisson's ratio = 0.3 for steel
R = radius of pipe, in.

To provide for slight imperfections and other variations from ideal conditions, design for the critical pressure divided by 2. This results in the following estimated design pressure limit:

$$P_e = \frac{3EI}{2(1 - \mu^2) R^3}$$

where: P_e = design pressure limit, psi, including a factor of safety of 2.0

These equations assume the pipe is round and surrounded by a uniform pressure fluid. However, no pipe is perfectly round and the strength reduction due to an out-of-round condition is significant as shown in Figure 8.4. Thus, if the deflection exceeds 1 or 2 percent, a further reduction in the design pressure may be in order.

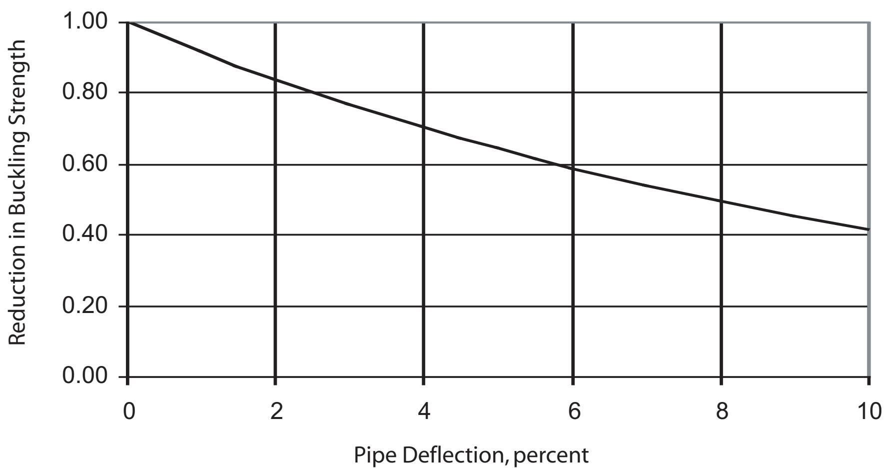


Figure 8.4 Reduction in hydrostatic buckling strength due to pipe deflection.

Many applications such as using CSP as a vertical standpipe in a lake, grouting it in as a vertical shaft liner, etc. do not subject the pipe to uniform pressure. Rather, the maximum pressure occurs at the lowest elevation, which is often imbedded in concrete for support. If the critical buckling pressure is reached near the level of the concrete, the pipe cannot buckle because it is supported by the immediately adjacent concrete. Additionally, a few feet above this point, the pipe is exposed to a lower pressure. To some degree, this portion of the pipe with less pressure provides additional support against buckling for the critically loaded section.

RELINE STRUCTURES

Corrugated steel pipes and structures are widely used for rehabilitation and reline applications as discussed in Chapter 12. The ability to manufacture CSP in any diameter necessary as well as to supply it with a hydraulically smooth interior adds to its appeal.

Rehabilitation methods typically involve grouting the annulus between the liner and the host structure. Grouting in this manner often stops the deterioration of the host pipe while it adds several inches of structural grout. Typically, reline structures are designed to carry the entire load imposed on them. This is inefficient in that an in-service host structure has at least a factor of safety of 1.0 under those same loads. However, design can conform to methods reviewed in Chapter 7, with the exception that the stated flexibility factors are not applicable.

Careful, low-pressure grouting places lower bending stresses on the pipe than placing and compacting traditional backfill. Also, the reline structure usually can be braced against the host structure to prevent flotation and improve buckling strength. The stiffness (flexibility factor limit) required in the liner becomes a matter of the grouting rate and technique involved. The Timoshenko buckling equation does not apply to grouting a horizontal structure. The pressure around it is not uniform. Rather the contractor must maintain a balance of grout depth, from side to side, much as would be done in using conventional backfill. The contractor must also handle buoyancy forces by bracing the reline structure off the host pipe.

For construction details, see Chapter 10.

VERTICAL SHAFT LININGS

Vertical shafts are used to construct inspection pits, deep foundations, insertion pits, etc. Often these are temporary structures that are used as forms or construction aids and not as permanent structures. In stable ground conditions, where a bored hole will stand for a short period, shaft linings are often picked up and inserted into the hole in a single piece. Alternatively, where ground conditions demand, shafts are excavated from the surface in stages with segmented 2-flange steel liner plate linings being erected in the hole as it advances. Corrugated steel pipe, structural plate, and steel liner plates, are typically the material of choice for shaft liners.

The loads on vertical shaft linings are quite different than those on normal buried pipes. Soil loads typically are limited to the active soil pressure acting on the shaft. These can be as little as a third or half of the soil prism load carried by a buried pipeline at the same depth. Once a shaft penetrates the water table, the liner must carry both the full hydrostatic load of the water and the active pressure of the buoyant soil.

While the design example that follows suggest a factor of safety of 2, the actual requirement for temporary structures depends on how well the ground conditions (and resulting loads) are known and the necessary degree of safety to protect the bore and any workers involved. In some instances, lower factors of safety may be acceptable. Design for the imposed loadings is identical to those for manhole shafts, but drag-down loads can be ignored when the bore is in stable soil that will not settle.

The necessary installation stiffness must be addressed. Vertical shafts can typically be more flexible than a buried pipeline since the installation loads are less severe. However, if the liner is to be back grouted rapidly, the resulting fluid grout (hydrostatic) pressure may dictate the necessary stiffness. The Timoshenko buckling equation and discussion, should be reviewed. With the many corrugated steel alternatives, the contractor can select an appropriate minimum stiffness to meet the installation requirements of the site and construction sequence.

A 16 foot diameter structural plate shaft liner is picked up to be inserted in a bored shaft.

Inserting the structural plate shaft liner.

Unlike backfilling a conventional pipeline or grouting a reline structure, grouting the annulus of a vertical shaft induces uniform, radial loads around the liner. Thus, the compaction and unbalanced fill loads induced during conventional pipeline installation are avoided. Unlike a conventional (horizontal) tunnel liner or reline structure, the vertical shaft does not need to support an unbalanced, side-to-side, grout loading during construction.

Other construction loads can come from a slough-in or other soil failures and surface loads from construction equipment, etc.

Safety is the major consideration. Even where workers are not in the hole, the cost of losing the bore is a consideration the contractor must address. Increasing the stiffness of the liner can provide an added measure of safety if a slough-in occurs. The effectiveness of additional stiffness depends on the specific site conditions, construction practices and other factors difficult to predict.

Typically, the stiffness requirements for shaft and tunnel liners is expressed as:

$$\text{Minimum Stiffness} = EI/D^2 \geq \text{Stiffness Factor}$$

The contractor or his engineer should provide stiffness limits. Where they are not provided, suggested stiffness factors for vertical shafts are summarized in Table 8.2. Lower stiffness liners may be used, depending on ground conditions and construction practices.

Table 8.2

Typical stiffness factors for vertical shaft liners

Corrugation Depth (in.)	Stiffness Factor (lb/in.)
1/2, 3/4 & 1	17
2	33
2-flange liner plate	33
4–flange liner plate	74

Design Example

Given: Shaft diameter = 12 ft
Excavation depth = 38 ft
Water table depth = 25 ft
Soil Unit Weight, γ = 120 lbs/ft^3
Buoyant unit weight γ' = 72 lbs/ ft^3
ϕ = 30 degrees
Active earth pressure, K_a = Tan (45-ϕ/2) = 0.333
Liner will be grouted in place.

Solution:

1. Design Pressure:
   Earth pressure at 25 feet = γ K_a 25 = 999 psf
   Buoyant earth pressure at 38 ft = γ' k_a (38-25) = 312 psf
   Water pressure at 38 feet = 62.4 (38-25) = 811 psf
   Total pressure P = 2122 psf

2. Ring Compression:
   C = P x S/2
   Where S = span, ft
   Then C = (2122) x (12/2) = 12,732 lbs/ft

3. Allowable Wall Stress
   For 12 ga 2-flange liner plate
   S= 144 <(r/k) $[24E/F_u]^{.5}$ = 366 in.
   where:
   A wall = 1.62 in^2/ft, I=0.049 $in.^4/in.$, r = .602,
   SS = seam strength= 30,000 lbs/ft
   K = 0.22 assumed for grout backfill
   E = 30,000,000 psi
   F_u = 42,000 psi
   S is less than 366 in., therefore:
   $F_b = F_u - [(F_u^2/48E)]\ (kS/r)^2$ = 38,607 psi
   $F_b > F_y$, Therefore: F_c= 28,000 psi and $f_c = F_c/2$= 14,000

4. Required Wall Area
   A = C/f_c = 12,732/ 14,000 = 0.909 in^2/ft < 1.62
   12-ga. 2-flange steel liner plate **OK**

5. Seam Strength
   SS ≥ 2C = 2(12,732) = 25,464 lbs/ft < 30,000 lbs/ft
   12-ga. 2-flange steel liner plate **OK**

6. Minimum Stiffness
   Min. Stiff = EI/S^2 = 70.8 > 33
   12-ga. 2-flange steel liner plate is **OK**

7. Ultimate Buckling Pressure due to fluid grout
   $P_{cr} = 3EI/[(1-\mu^2)R^3] = 3(30000000)\ 0.049/[(1 - 0.3^2)\ 72^3] = 13.0$ psi

   where:
   P_{cr} = critical buckling pressure, psi
   μ = Poisson's ratio = 0.3 for steel
   R = pipe radius, in.

Equivalent feet of fluid grout = 13.0(144)/140= 13.4 ft (A suitable factor of safety needs to be applied).

TUNNEL LINERS

Loads

When steel structures are installed by jacking or tunneling, the soil load on the structures may be considerably less than indicated by the load factors, K, discussed in Chapter 7. In sound soils, the jacking or tunneling processes can produce a soil bridging effect that keeps most of the load off the structure. However, in the case of plastic clays, the entire soil overburden load is likely to come to rest on the structure at some point in time.

During the first half of the 20th century, Anston Marston participated in numerous load studies and developed the soil load theory on buried structures that is still widely used today. It forms the basis for the current loading charts for steel tunnel liner plates. The design pressure acting at the top of the tunnel or jacked pipe structure can be taken as:

$$P_d = C_d\,(\gamma H + PL)$$

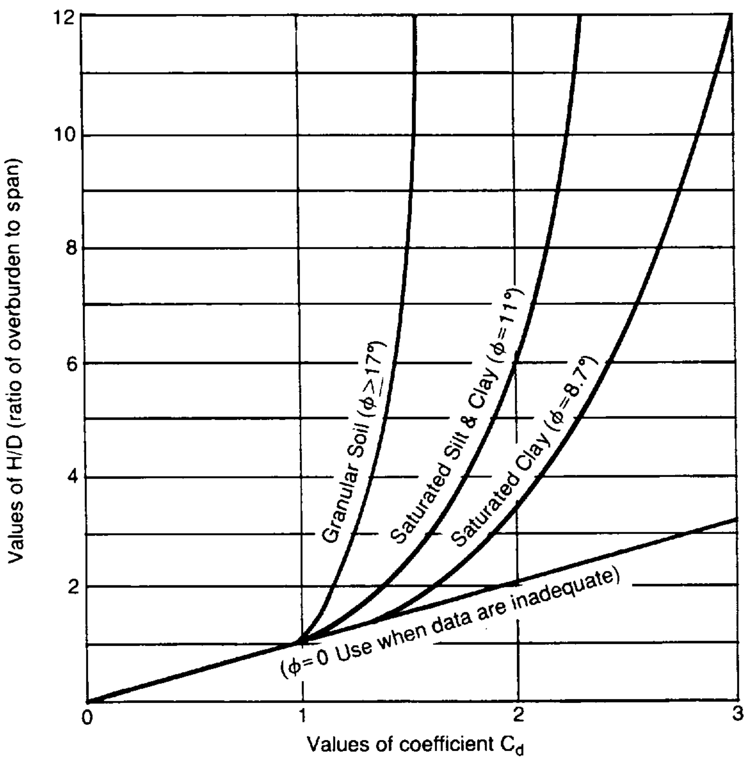
Where:
C_d = soil load coefficient from Figure 8.5
C_d = 1.0 if inadequate information is available to describe the soil at the level of the structure.
γ = soil density, typically taken as 120 pcf
PL = live load pressure (from Table 7.7) taken at the crown elevation of the structure.

Design considerations

Once the design pressure (P_d) is determined, it is used to calculate the thrust in the structure and design checks are otherwise performed in accordance with Chapter 7. The soil stiffness factor K used in buckling calculations depends on the backfill or grout immediately around the structure. For a liner backfilled with clay, K, is taken as 0.44 while sand backfilling produces a design value of 0.22. Typically, a grouted annulus is checked using a K of 0.22 even though this can be very conservative.

Having determined the actual load on the tunnel liner, the remainder of design follows principles in Chapter 7, to achieve a minimum factor of safety of 2.0. However, construction stiffness can become an issue. The stiffness of liner plate is calculated as the reciprocal of its flexibility factor. That is, the minimum stiffness for a horizontal, 2-flange tunnel is:

$$\text{Minimum Stiffness} = EI/D^2 \geq 50 \text{ lb/in.}$$



■ **Figure 8.5** Diagram for coefficient C_d for tunnels in soil (ϕ = friction angle)

Where: E = young's modulus for steel = 30,000,000 psi
I = tunnel liner moment of inertia (from Table 8.3)
D = diameter of the tunnel liner, in.

A minimum stiffness value of 50 lb/in. is equivalent to a flexibility factor limit of 0.020 inches per pound used for an embankment installation of a 2 inch deep corrugated steel structural plate. It provides adequate stiffness for backfilling (done by compacting the void between the liner and bore full of sand) or grouting.

Unlike a vertical shaft liner, a tunnel liner is often selected to provide a higher stiffness than needed for simple backfilling. Until it is supported by grout or backfill, the liner is protecting the crew from collapse of the tunnel bore. In poor soils or instances when the

Table 8.3

Structural properties for 2-flange liner plate*

Thickness (in.)	Effective Wall Area ($in.^2/ft$)	Effective Moment of Inertia ($in.^4/in.$)	Ultimate Seam Strength (lb/ ft)
0.075	1.152	0.034	20,000
0.105	1.620	0.049	30,000
0.135	2.088	0.064	47,000
0.164	2.556	0.079	55,000
0.209	3.264	0.103	87,000
0.239	2.868	0.118	92,000

Notes:
* Steel per ASTM A 569
Tensile strength = 42,000 psi
Yield strength = 28,000 psi
Minimum Elongation (2 in.) = 30%

actual soil conditions have not been determined, one often selects a heavier gage steel tunnel liner to obtain two or three times the minimum stiffness requirement to provide added protection for the crew.

AERIAL SPANS

Should the need arise to run water or sewers, etc. above ground or under bridges, CSP aerial sewers supported on bents afford an economical solution. Table 8.4 provides allowable spans for this purpose. The table is for pipes flowing full of water, including the weight of an asphalt-coated pipe. The bending moments were calculated on the basis of a simple span and with the pipe bending strength determined by limited testing.

Consideration must be given to the design of the pipe support system. Small diameter pipe with short spans can often be placed directly on bents. Larger diameter pipe should be supported by shaped, 120-degree concrete cradles or by a ring girder. The importance of the support requirements increases with diameter and span. Design methods used for smooth steel water pipe systems can be adapted to investigate these requirements.

COLUMN OR END LOADS

Tests were conducted as early as 1930 at the University of North Carolina and the University of Illinois to determine the strength of 2-2/3 x 1/2 inch corrugated steel pipe for carrying compression end loads. These results are useful in determining the necessary strength of circumferential seam connections, maximum jacking loads and its strength for use as bridge piers, caissons, vertical shaft liners and other columns used in construction.

Corrugated steel pipe aerial sewer.

Table 8.4

Allowable span in feet for CSP flowing full

Diameter of Pipe, in.	Specified Steel Thickness, in.								
	0.064	0.079	0.109	0.138	0.168	0.188	0.218	0.249	0.280
2-2/3 x 1/2 in. Corrugation									
24	13	15	20						
36	12	15	20	25					
48	11	14	19	25	30				
60		14	19	24	29				
72			18	24	29				
84				23	28				
96					27				
5 x 1 in. or 3 x 1 in. Corrugation									
36	9	11							
48	9	11	15						
60	8	10	14	18					
72	8	10	14	18	22				
84	8	10	14	18	22				
96		10	14	18	22				
108			14	18	21				
120				17	21				
6 x 2 in. Corrugation									
			0.111	0.140	0.170				
72			12	15	17	19	22		
84			11	14	17	19	22	24	27
120			11	14	16	18	21	24	27
144			11	13	16	18	21	24	27
168			10	13	16	18	21	23	26
192			10	13	16	17	20	23	26
216				12	15	17	20	23	26
240					15	17	20	22	25

Subsequent tests at the Ohio State University (1965) confirmed that these short column results are conservative for both annular and helically corrugated steel pipe. Ultimate short column or compression block values for the 2-2/3 x 1/2 inch corrugation are provided in Figure 8.6.

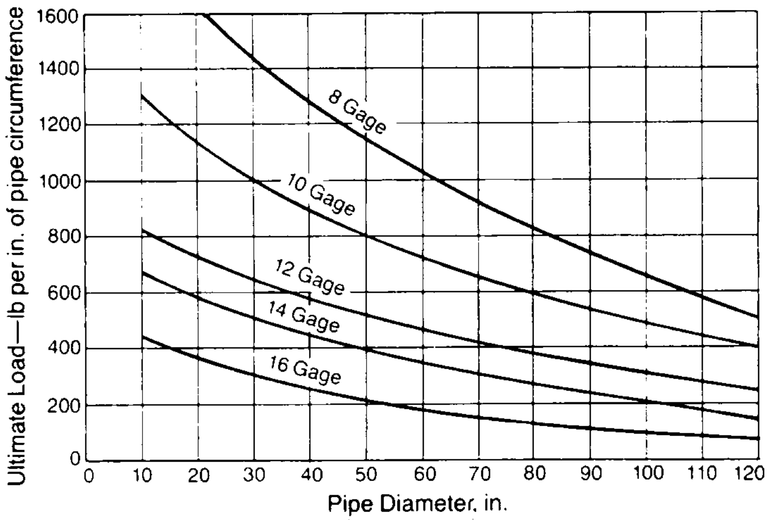


Figure 8.6 Ultimate unit compressive strength of short 2-2/3 x 1/2 inch corrugated steel pipe columns as determined at University of Illinois.

Recent strength testing, comparing the strength of other corrugations with 2-2/3 x 1/2 extends this earlier work to a broader range of corrugations. Being shallow, the 2-2/3 x 1/2 corrugation has more column load capacity than the deeper corrugations. Values tabulated in Table 8.5 are from testing done to develop ASTM A 998 and are expressed as a multiplier to reduce the values for 2-2/3 x 1/2 end loads from the figure.

Table 8.5

Column or end load strength

Corrugation Depth (in.)	Factor
3/4 (rib)	0.30
1	0.44
2	0.30

Example

Given: Determine the end load strength of a 54 in. diameter, 0.064 in. thick, 3 x 1 in. pipe

Solution

From Table 8.5 the end load capacity of 3 x 1 in. corrugated steel pipe is 44% that of a 2-2/3 x 1/2 in. pipe.

From Figure 8.6 the end load strength of 54 in. diameter, 0.064 in. thick (16 gage) 2-2/3 x 1/2 in. = 200 lbs per circumferential in.

Thus the end load capacity of 54 in. diameter 3 x 1 in. = 0.44 (200)= 88 lbs per circumferential in.

RECLAIM (CONVEYOR) TUNNELS

Reclaim tunnels and conveyor enclosures are nearly horizontal, buried pipe applications. However, they have special features that typically include a varying dead load caused by a rising and falling ore pile, as well as ore hoppers and conveyor bents that are often hung off the structure. At the same time, when the storage piles are run up, most of these structures are near their maximum safe cover limit, while they are often at minimum cover when the last of the ore is being bladed into the hoppers.

Because of their critical nature, they must be properly designed and the best quality backfill is needed. Yet a load reduction factor ($K<1.0$) does not apply to these structures because the loading is cyclical. Although load relief occurs the first time the ore pile is built up, additional deflection would need to occur each subsequent time in order to continually achieve load relief. In fact, these structures become locked in to the backfill and do not continue to deflect with each load application.

When the ore is drawn down it is not uncommon to use a front-end loader to push the last of the ore into the hopper. Compacted structural backfill needs to continue up a distance of span/8 above the structure, but if the working axle load of the loader exceeds H20, additional cover will be required (see Construction Loading, Chapter 10).

The hopper openings in the structure must be reinforced. While procedures are similar to reinforcing fittings in pipe, the openings here do not have a fitting stub welded integrally into the opening. Therefore, the reinforcement becomes more significant than that outlined in ASTM A 998. The fabricator should provide the reinforcement details, which are typically sized to carry the bending moment developed in the longitudinal reinforcements due to the thrust in the pipe. Assuming a simple beam, this results in a bending moment, M, in.-lb, of:

$$M = (C\, l^2)/8$$

Where: C = ring compression in the pipe (lbs/in.)
L = unsupported length of the reinforcement (in.)

Finally, the ore hoppers and conveyors are often hung directly off the structure. These loads not only put extra thrust in the structure, they typically are point loads applied to the structure asymmetrically. The hoppers and conveyor loads should not be hung directly. Rather stiff, curved, ring beams are typically applied to the outside of the structures so they bear, much like a saddle, on the corrugated steel structure. Ring formulas can be used to evaluate their necessary length and stiffness.

Alternatively, light crane loads as well as ore hopper and conveyor loadings can be supported by using stiff longitudinal beams to spread the loads sufficiently along the length of the structure. The longitudinal beams should be much stiffer than the longitudinal stiffness of the structure so the beams spread the point loads over enough attachment points for the structure to carry the loads.

MINIMUM COVER EVALUATIONS

Minimum heights of cover are difficult to calculate directly. Minimum cover levels have been determined through experience with the primary concern being that of maintaining the pavement, not collapsing the pipe.

To date, the best analytical approach to minimum cover requirements has been developed by Dr. J. M. Duncan. This method accounts for the size of the axle load, the plastic moment strength of the pipe and the stiffness of the backfill. However, it is not completely calibrated and if the entire method is applied it may not provide results that agree with the accepted, experience-based minimum cover limits.

A simplified approach based on Duncan's work can be used to see the effects of increasing axle loads, heavier than normal steel thicknesses, or improved backfill. A constant C_3, below, is calculated based on corrugation, axle load and soil stiffness. The C_3 value can be used to provide a ratio of the necessary minimum cover depths or plastic moment strength (M_p) requirements from the AASHTO Span / 8 (Span / 4 for Spiral rib) rule for the specific pipe involved. Consider the following:

$$M_p = C_3(S/H)^2 \quad \text{or} \quad H = S/(M_p/C_3)^{0.5}$$

where:

S = span of the structure (in.)
H = actual minimum cover (in.)
C_3 = constant such that:
= 69 AL/(32C)

C = 69 for 90% standard Proctor compaction, or
C = 115 for 95% standard Proctor compaction
AL = Axle load (kips)
M_p = plastic moment strength of the pipe wall (ft-kips/ft)

From the suggested value of C_3 it is seen that the calculation for M_p assumes a 32 kip axle load along with 90% Proctor density backfill. Thus it can be seen that doubling the axle load either doubles the plastic moment strength required in the pipe or increases the necessary depth of cover by $(2)^{0.5}$. Using 95% standard Proctor density backfill in lieu of 90% Proctor reduces the necessary plastic moment strength to 60% (69/115 = 0.6) of that originally required. A similar increase in density of the backfill can reduce the required minimum cover height to about 80% of the AASHTO level ($0.6^{0.5}$). This method will provide a ratio to change the standard fill height requirement to an actual minimum cover needed for heavier axle loads by accounting for the benefit of stiffer backfill or a heavier gage (higher M) pipe. It is conservative to use a heavier axle load if that larger load leads to a wider footprint than the 20 square in. commonly assumed for an H 20/H 25 dual tire load. However, it is not recommended that a ratio be used for the minimum cover required for a lower axle load when the footprint is reduced.

To account for increasing the soil density from 90 to 95% Proctor, it is most reliable if a crushed rock backfill or a clean A1 material (course sand or gravel) per AASHTO M 145, is specified and its density field tested. Using select backfill materials help obtain the necessary soil stiffness.

This approach is not intended to replace the standard, experience-based, minimum covers from Chapter 7. However, it can be used to evaluate field conditions where grade elevations do not meet design requirements.

One use of the ratio allows accounting for too little minimum cover by increasing the material thickness and its plastic moment strength (M_P). In doing so, however one must recognize that a minimum cover of span / 10 requires a higher theoretical factor of safety than span / 8. It is suggested the required Mp be increased by an additional 10% as cover decreases to a minimum of span / 10.

For the sake of conservatism, changes should be limited to no more than one or two parameters in the design. Generally, minimum covers should not drop below span /10 (span / 5 for spiral rib pipe) or the 12 inch minimum not reduced to less than 9 or 10 inches.

Beyond moment strength considerations, high shear strength backfill materials such as crushed rock, cement stabilized sand or cement slurries have long been used to reduce the required cover on a steel pipe installation. Incorporating these materials can reduce the required minimum cover to 67% of the original requirement. Double reinforced structural concrete has been used as a load relief slab or saddle to reduce the required minimum cover by as much as 50%.

Table 8.6

Plastic moment strengths "M_p" of corrugated steel pipe (k-ft/ft)

Wall Thickness (in.)	2-2/3 x 1/2	3 x 1 / 5 x 1	6 x 2	15 x 5-1/2**	16 x 6**
0.064	0.39	0.79			
0.079	0.49	0.99			
0.109 /0.111*	0.69	1.40	2.66		
0.138/ 0.140*	0.90	1.82	3.44	14.43	
0.168/ 0.170*	1.11	2.24	4.22	17.66	18.46
0.188			4.73	19.75	
0.197					21.70
0.218			5.54	23.07	
0.236					26.34
0.249			6.36	26.39	
0.276					30.78
0.280			7.18	29.72	
0.315					35.00

Notes:
* Where two thicknesses are provided, the first is for pipe and the second for structural plate.
** M_p values for 15 x 5-1/2 and 16 x 6 corrugation are based on a yield strength (Fy) of 44 ksi.

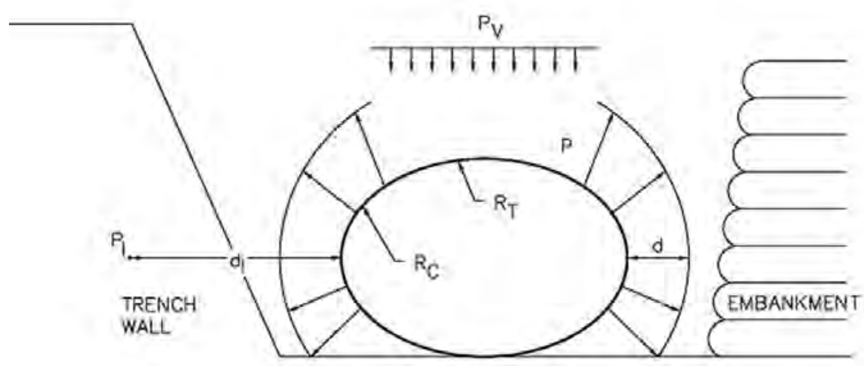
BEARING PRESSURE EVALUATIONS

Chapter 7 provides a means of evaluating bearing pressures at the tight radius corners of pipe arch, ellipse and underpass shapes. Typically, bearing pressures are calculated at the surface of the steel structure and a nominal width of backfill is provided. However, under high covers or in soft soil conditions, it becomes desirable to evaluate these pressures at various distances from the pipe to determine the effect of a wider backfill zone or the necessary strength of the embankment or trench wall.

For example, the designer may want to limit the horizontal compression strain of the backfill and embankment outside of it to 1% of the pipe diameter. Theoretically at least, this would result in 2% increase in span due to compression in the soil on both sides of the pipe. To make this evaluation, the designer may elect to calculate the pressure at the center of the backfill zone and at one or two distances out into the embankment or trench wall beyond. At a distance of one span from a round pipe the pressure in the soil has generally returned to its at rest pressure, even with the pipe in place.

Forces acting radially off the small radius corner arc of the structure at a distance d_1 from the plate surface may be taken as:

$$P_1 = T \,/\, (R_c + d_1)$$



■ **Figure 8.7** Bearing pressure evaluations.

Where: P_1 = horizontal pressure from the structure at distance d_1 (psf)
d_1 = distance from the structure (ft)
T = total dead load and live load trust (lb/ft)
R_c = corner radius of the structure (ft)

The required backfill envelope width, d, to limit strain in the trench wall or embankment is:

$$d = (T/P_{brg}) - R_c$$

Where:
d = required backfill envelop width (ft)
P_{brg} = allowable bearing pressure to limit the compressive strain in the trench wall or embankment to a suitable level (psf)

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Installation of polymer coated CSP.

CHAPTER nine

Durability

INTRODUCTION

Corrugated steel pipe (CSP) has been used successfully since the late 1800s for storm sewer and culvert applications throughout North America and around the world. CSP continues to be specified because of the variety of CSP products and their ability to provide long service life in a wide variety of site environmental conditions. Durability describes the ability of a specific material to resist degradation caused by corrosion, abrasion, applied loads, and method of installation. Throughout the long history of CSP, more than 50,000 installations have been the subject of critical evaluation to establish durability guidelines. The behavior of both the soil side and the water side of the pipe has been studied. These studies have shown that CSP can provide outstanding durability and that virtually any required service life can be attained by selecting the appropriate coating and thickness of steel for the pipe wall.

This chapter explains how the methods of evaluating pipe durability have evolved with expanded experience, new materials and knowledge of the impact of site conditions. The results of field studies, along with methods for predicting the durability of CSP in differing conditions, are addressed. Various metallic and nonmetallic coatings, various steel thicknesses, and where necessary invert pavements, can be used to enhance the durability of CSP and provide the necessary service life.

SCOPE

This chapter provides the designer with information and methods needed to evaluate the durability (service life) of corrugated steel pipe products. A vital part of this chapter is a brief review of the significant field surveys, inspections and studies that have provided the historical data needed to improve the tools and methods used to estimate the service life of CSP.

Over the past 50 years, corrugated steel pipe has gone through an evolutionary process with the result being better materials, with longer service life in challenging environments. One of these materials is Aluminized Type 2, which has been in service at thousands of sites since 1948. When installed in the recommended environmental range, Aluminized Steel Type 2 CSP will have a minimum service life of 75 years.

A coating developed in the early 1970's is a 10 mil polymer film that is laminated over galvanized steel. When installed in the recommended environmental ranges, this coating will have a service life of over 100 years. Polymer coated pipe can be installed in conditions more severe than concrete pipe and have a longer service life. Inspections of polymer coated pipe in service for over 35 years, in a range of conditions, show no deterioration.

Table 9.1

Estimated Material Service Life for CSP

CSP Material	Estimated Service Life	Site Environmental Conditions	Maximum FHWA Abrasion Level
GALVANIZED CSP	AVERAGE 50 YEARS	$6.0 \leq$ pH ≤ 10.0 $2000 \leq r \leq 10{,}000$ (ohm-cm) Water Hardness (> 50 ppm $CaCo_3$)	LEVEL #2
ALUMINIZED TYPE 2 CSP	MINIMUM 75 YEARS	$5.0 \leq$ pH ≤ 9.0 r > 1500 ohm-cm	LEVEL #2
POLYMER COATED CSP*	MINIMUM 100 YEARS	$5.0 \leq$ pH ≤ 9.0 r > 1500 ohm-cm	LEVEL #3
	MINIMUM 75 YEARS	$4.0 \leq$ pH ≤ 9.0 $r \geq 750$ ohm-cm	
	MINIMUM 50 YEARS	$3.0 \leq$ pH ≤ 12.0 $r \geq 250$ ohm-cm	

NOTE: Refer to Table 9.3 for definition of FHWA abrasion levels.
* Polymer coating is 0.010 in. on each side.

The method for estimating service life is different for galvanized than for Aluminized Type 2 or polymer coated CSP. This chapter first defines and explains the method used to determine service life for galvanized CSP. Evaluating all three coatings will ensure the pipe materials specified for your project will meet service life requirements at the lowest cost.

SERVICE LIFE DEFINITIONS

Design Service Life

Many public agencies establish a design service life (DSL) for construction of infrastructure projects. The DSL for roadway projects is dependent upon the type of roadway, the traffic volumes, and future growth patterns. Depending on the agency involved, the DSL of roadways typically varies from 25 to 100 years.

Estimated Material Service Life

Estimated Material Service Life (EMSL) is defined as the years of reliable service from the time of installation until rehabilitation or replacement is required. The EMSL of a pipe is dependent on the pipe material and environmental conditions. Table 9.1 shows the EMSL of CSP with three different coating systems.

It is not necessary that the EMSL of a pipe match the DSL of the project. For example, pavements, bridge decks and other portions of a highway are typically replaced or rehabilitated several times during the life of that highway. CSP can be rehabilitated by sliplining, paving the invert or other methods. Many agencies now oversize the initial pipe installation to accommodate future rehabilitation. Most agencies routinely inspect all culverts, regardless of the pipe material, to ensure timely identification of problems.

The material selection for a culvert or storm sewer should recognize the life-cycle economics including the costs of the initial installation, routine inspection, maintenance, and possible rehabilitation (see Chapter 11). Numerous evaluations of the durability of storm sewers and culverts show that the EMSL of storm sewers can be expected to be greater than for culverts, due to lower flow velocities, intermittent flow, and reduced bed load size and quantity.

FACTORS AFFECTING THE DURABILITY OF CSP

Durability In Soil

The material in the soil envelope around the pipe or backfill, is generally controlled by the project specifications. The better suited the backfill material is for structural support of the pipe, the less corrosive it is likely to be toward the pipe. Table 9.2 compares the corrosiveness of several soil types. In corrosive soils, an envelope of properly specified backfill material around the pipe can protect the pipe from most of the corrosive elements inherent in the native soil. However, soil-side corrosion is rarely the determining factor in predicting pipe service life.

The durability of steel pipe in soil is a function of several interacting parameters including soil resistivity, acidity (pH), moisture content, soluble salts, and oxygen content (aeration). All underground corrosion processes involve the flow of current (conductivity) through the ground from one location to another (a corrosion cell). Resistance to current flow through a material is measured by the resistivity of that material. The greater the resistivity, the smaller the current flow, which tends to lower the corrosion rate. The resistivity value, expressed as ohm-cm, is inversely proportional to the conductivity value.

The pH value of a substance is a measure of the hydrogen ion concentration in the substance. Most soils fall within a pH range of 6.0 to 8.0, which is considered to be the neutral range and is favorable to the durability of steel pipe. Soils with lower pH values (acidic soils), usually found in areas of high rainfall, tend to be more corrosive and require a more careful selection of the pipe coating(s). The pH values of soil and water at a site are significant factors in selection of the right pipe product.

The moisture content in the soil can also be a significant factor affecting the durability of CSP. Granular soils that allow rapid drainage of the pipe backfill enhance durability. Soils with moisture content less than 20 percent tend to be non-corrosive to CSP on the soil-side. Soils with high clay content tend to hold water longer and therefore are more corrosive than well drained soils.

Table 9.2

Corrosiveness of Soils

Soil Type	Description	Aeration	Drainage	Color	Water Table
I. Non Corrosive	1. Clean Sands 2. Well graded gravel	Excellent	Excellent	Uniform	Very low
II. Lightly Corrosive	1. Sandy loams 2. Light textured silt loams 3. Porous loams or clay loams thoroughly oxidized to great depths.	Good	Good	Uniform color	Very low
III. Moderately Corrosive	1. Sandy loams 2. Silt loams 3. Clay loams	Fair	Fair	Slight mottling	Low
IV. Badly Corrosive	1. Clay loams 2. Clay	Poor	Poor	Heavy texture Moderate mottling	2 to 3 ft below surface
V. Unusually Corrosive	1. Muck 2. Peat 3. Tidal Marsh 4. Clays and organic soils	Very poor	Very poor	Blue, gray, green	At surface; or extreme impermeability

NOTE: Soil types III, IV and V are poor quality and are not recommended for use as backfill.

Durability In Water

The water side of the pipe is typically subjected to additional detrimental actions that are more severe than those acting on the soil side. Field studies have shown that the portion of the pipe most susceptible to corrosion is the invert. This should not be surprising since the invert is exposed to standing or flowing water more than any other part of the pipe interior. Common factors affecting durability are pH, resistivity, soluble salts, water hardness and abrasion.

Most storm or natural waters will have a pH in the range of 6.0 to 8.0. The chemistry of the water is controlled by rainfall which leaches salts from the soil on its way to the culvert or storm sewer. These chemicals are in the form of soluble and partially soluble salts. Groundwater may also contain various dissolved salts removed from the soil itself. These dissolved salts can contribute to corrosion by increasing the conductivity of the ground water and thus lowering the soil's resistivity. Conversely, many salts in the soil form layers of carbonates or hydroxides on the coating surface. These layers of chemical compounds have the effect of reducing corrosion. However, high levels of other salts, principally chlorides and sulfates will make a soil more aggressive.

In the presence of hard waters ($CaCO_3$ > 50 ppm), a process called scaling results in the deposition of a protective barrier on the pipe surface and reduces corrosion. While increasing amounts of $CaCO_3$ protect the pipe, increasing levels of dissolved oxygen and CO_2 can accelerate corrosion. The most important effect of increased levels of CO_2 in water relates to its interference with the formation of the protective $CaCO_3$ scale that develops on galvanized pipe surfaces. High resistivity levels in water (R > 10,000 ohm-cm) may indicate soft water ($CaCO_3$ < 50ppm). Soft water has a reduced ability to neutralize acid events often attributed to air pollution and acid rain. This condition in combination with minimum thickness of protective scale is conducive to accelerated corrosion rates in galvanized steel.

The Aluminized Type 2 (ALT2) coating performs well in soft water where the oxygen present in the water forms an aluminum oxide layer on the pipe wall that extends the pipe's service life. The laminated film over the galvanized steel on polymer coated pipe provides an inert protective barrier in soft water and severe environments.

The CSP involved in establishing the charts used in the earlier performance charts developed by CALTRANS, AISI, FHWA and others, was galvanized steel pipe installed 40 to 80 years ago, since this was the only CSP available during that time period. This resulted in galvanized CSP being installed in sites that likely would not be environmentally appropriate for galvanized CSP today. Water hardness, a parameter that has a significant impact on the service life of galvanized CSP, was not measured at the culvert sites. Had the impact of soft water on galvanized CSP been recognized at the time of installation of the CSP evaluated in these early studies, the current methods used to estimate the service life of CSP would result in longer service life predictions, for galvanized CSP installed within the environmental guidelines detailed in this chapter.

Durability In Abrasive Conditions

Abrasion is the removal of the coating and the deterioration of the steel from the invert of the pipe wall caused by high velocity water and abrasive material the water carries with it. Protective barrier layers or scaling will improve performance in abrasive conditions. Table 9.3 provides a classification of abrasive conditions, established by the FHWA, based on flow velocity and bed load.

Storm sewers tend to have flatter slopes, lower velocities, and are exposed to smaller, less abrasive bed load material than culverts. Hence, storm sewers typically experience few abrasion issues. Culverts generally have steeper slopes, higher velocities, continuous flow, and more significant bed loads.

Table 9.3

FHWA Abrasion Levels

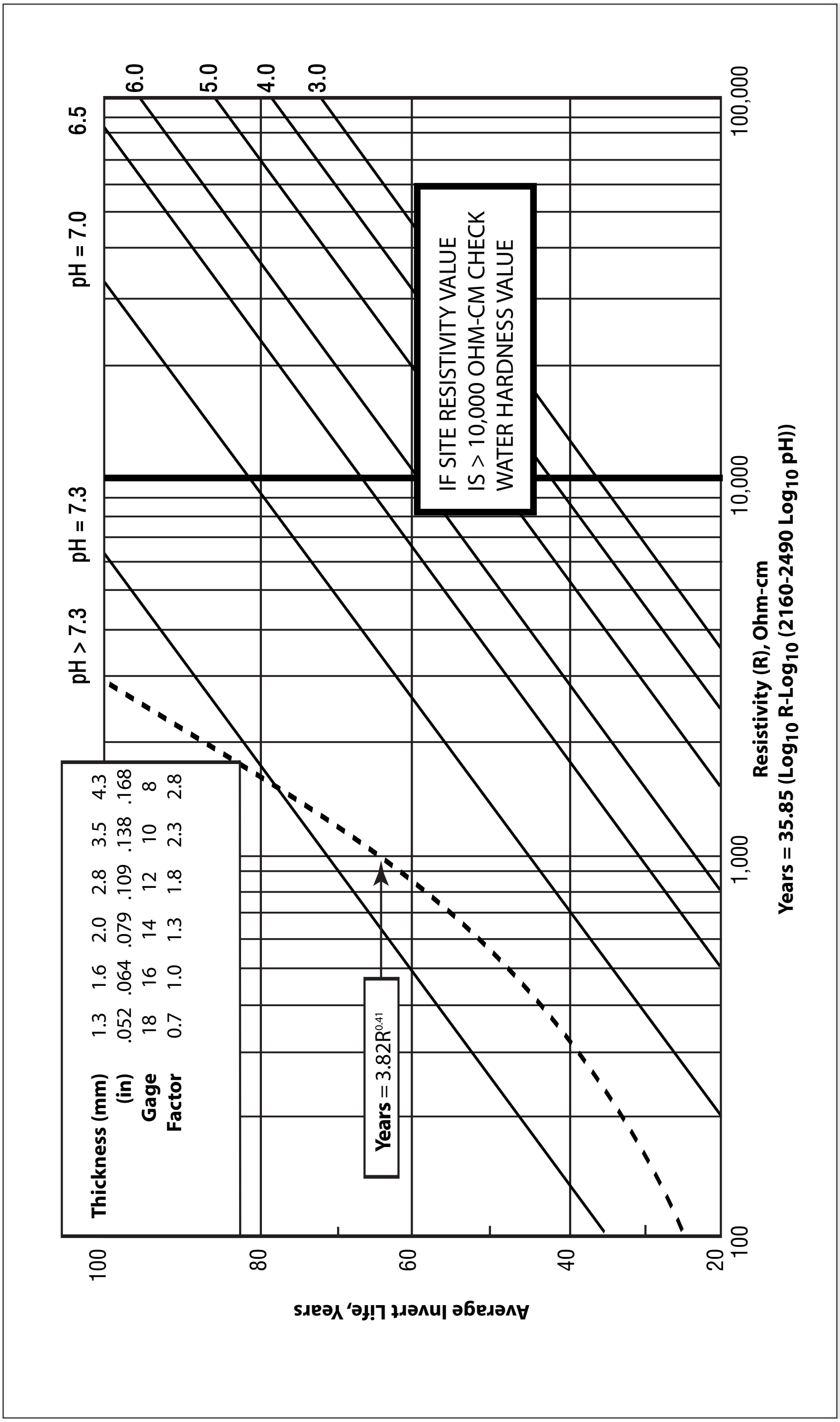
Level 1	Non-Abrasive	No bed load regardless of velocity; or storm sewer applications
Level 2	Low Abrasion	Minor bed loads of sand and gravel and velocities of 5 ft/sec. or less
Level 3	Moderate Abrasion	Bed loads of sand and small stone or gravel with velocities between 5 and 15 ft/sec.
Level 4	Severe Abrasion	Heavy bed loads of gravel and rock with velocities exceeding 15 ft/sec.

NOTE: Consideration of velocities should be based on a frequent storm event, such as a 2-year storm.

FIELD EVALUATION OF CSP DURABILITY

The best method for evaluation of the service life of CSP at proposed sites is evaluation of previously installed pipe. The first significant field evaluation of installed CSP was conducted by CALTRANS in the early 1960s. The data generated in the CALTRANS study of 7000 culverts was used to develop the chart for estimating average invert life of galvanized CSP (Figure 9.1). This figure predicts the EMSL of a pipe based on a loss of 25 percent of the steel in the pipe invert. This study evaluated the service life of CSP based on the values of pH and resistivity. Water hardness was not measured at the culvert sites. Since the CALTRANS field evaluation, numerous states in the US and provinces in Canada have conducted similar studies of their culverts. The results were wide spread with the variations due to a prevalence of soft water, heavy snowfall or possibly the heavy use of road salt. The results of regional studies emphasize the importance of using local information when available.

The predictive method developed by the CALTRANS evaluation depended on whether the pH exceeded 7.3. Where the pH was consistently less than 7.3, the resulting service life was controlled by the corrosion rate of the pipe invert, with the corrosion rate being dependent upon the combined influence of pH and resistivity. For sites with a pH greater than 7.3, soil-side corrosion was the controlling factor. These latter sites tended to be in the semiarid and desert areas with less than 10 inches of rainfall per year.. The CALTRANS report stated that at least 70 percent of the pipes were expected to last longer than indicated by the chart.



■ **Figure 9.1** Estimated material service life of galvanized CSP.

NCSPA/AISI Study

In 1986, the NCSPA, with the cooperation of the AISI, commissioned Corrpro Companies, Inc., a corrosion consulting firm located in Medina, Ohio, to conduct a condition and corrosion survey on corrugated steel storm sewer and culvert pipe. The installations investigated were located in 22 states scattered across the United States, and had installation durations ranging from 20 to 74 years. Soil resistivities ranged from 1,326 to 77,000 ohm-cm, and the pH ranged from 5.6 to 10.3.

The study showed that the soil-side corrosion was relatively minimal on most of the pipes examined. Where significant interior corrosion was observed, it was typically limited to the pipe invert. Specific predictive guidelines were developed on a statistical basis.

Invert pavements can be provided, by either factory or field application, to provide significant additional durability (Table 9.4). The data shows that galvanized CSP systems can be specified with a paved invert to provide a service life of 100 years in a variety of soil and water conditions.

New Coatings for CSP

The steel suppliers and the fabricators of CSP continued development of better performing coatings that would allow installation of CSP in environmental conditions that exceeded the environmental ranges favorable to galvanized CSP. Two coatings, Aluminized Type 2 and Polymer Coated CSP are the result of these efforts. The performance of these coatings has been verified by field evaluations conducted by many public agencies.

Aluminized Type 2

Aluminized Type 2 (ALT2) was introduced as an alternative coating in 1948. Extensive field studies of thousands of ALT2 pipe installations by industry, federal, state and Canadian agencies have confirmed its excellent performance. This product develops an oxide barrier layer on the interior pipe surface and thus performs well in areas where the presence of soft water would require a coating other than galvanized CSP. In addition to performing well in soft water conditions, CSP with an ALT2 coating performs in wider ranges of pH and resistivity. Continuing evaluation of CSP with the aluminized coating, installed at sites with the extended ranges of environmental conditions, has provided adequate evidence of the enhanced performance of this coating. When installed in the established environmental conditions, field evaluations of CSP with the ALT2 coating, have proven this material will achieve a minimum 75-year service life.

Polymer Coated

Further development by the CSP industry resulted in the introduction of polymer coated CSP in the 1970's. This coating was developed for use in environmental conditions that exceed those in which most materials can perform. This polymer coating is applied over the galvanized coating that was the standard coating for CSP for many decades. The presence of two coatings, and the resulting strong bond between these coatings, allows polymer coated CSP to achieve performance levels not attainable by most pipe materials and coatings. Field evaluations of polymer-coated CSP have proven that when installed at sites within the defined site environmental conditions, this pipe can provide service well in excess of 100 years.

DURABILITY GUIDELINES

CSP With Only a Galvanized Coating

The CALTRANS service life estimation method referred to previously was based on life to first perforation of an unmaintained culvert. However, the consequences of small perforations in culverts are usually minimal, and have little or no effect on the pipe's hydraulic or structural performance. The CALTRANS report stated that at least 70% of the pipes were expected to last longer than the life determined by using the CALTRANS chart. Given the ultra conservative results presented by the initial CALTRANS chart, the curves on the chart were converted by R. F. Stratfull to "average service life" curves, using data developed on weight loss and pitting of bare steel samples by the National Institute of Standards and Technology (NIST).

Experience gained from further field evaluations of galvanized CSP performance, has shown that sites with high resistivity values, might be indicative of soft water. This experience has led to further modifications of the CALTRANS chart. The latest change cautions the designer about the use of galvanized CSP on sites with high resistivities. It is recommended that sites with high resistivities be evaluated for water hardness before specifying the proper CSP product. Figure 9.1 shows the resulting chart for estimating the average invert service life of galvanized CSP, when the site pH and resistivity are known.

CSP With Supplemental Pavings Or Coatings

There may be sites where the three coatings available on steel delivered to pipe fabrication facilities may not be the best match for the conditions on the site. Additional add-on life can be provided by coating or paving the invert of CSP with asphalt or concrete either after fabrication or after installation. The number of years of additional life are independent of the base coating on the pipe and are shown in Table 9.4.

Table 9.4

Service Life Add-ons for Supplemental Pavings and Coatings

COATING MATERIAL	FHWA Abrasion Level 1	2	3	4
	Add-on Service Life (Years)			
Asphalt Coated	10	10	N/R	N/R
Asphalt Coated and Paved	30	30	20	N/R
Concrete Paved			80	80

NOTE: N/R = Not Recommended

EXAMPLES OF DURABILITY DESIGN

The following design examples indicate the proper methodology for selecting an appropriate CSP product from among the options available. The examples assume that the pH, resistivity, and abrasion level at the sites are known or can be estimated with reasonable accuracy. Also, it is assumed that the pipe has been hydraulically sized and structurally evaluated. The pipe initially selected is a 16 gage CSP with galvanized coating, which enables us to use Figure 9.1. If the 16 gage galvanized coating does not meet the DSL, the designer should consider a heavier gage galvanized pipe and one or more of the alternative coatings. It is possible the designer will find a number of alternatives that will meet or exceed the DSL. In this case, all pipe materials that satisfy DSL should be specified and the contractor allowed to make the final decision based on installed pipe costs.

The following examples assume that the water side of the pipe controls the durability design, i.e., the backfill materials and ground water do not create corrosive conditions that determine the pipe's EMSL.

Example No. 1

Site Conditions: pH = 6.5, resistivity = 4000 ohm-cm, abrasion = level 2
Water hardness = 200 ppm ($CaCO_3$)
Design Service Life = 50 years
Initial Pipe Selection (structural calculation): 48 in. diameter, 16 gage
(2 2/3 x 1/2 in. corrugation)

With r = 4000 ohm-cm, pH = 6.5, EMSL of galvanized pipe (Fig. 9.1) = 52 years (>50 years OK)

Water hardness > 50 ppm, (Water hardness not a problem; galvanized coated CSP can be used)

Alternative Pipe Selections:

See Table 9.1 for EMSL of polymer coated and ALT2.
Site environmental conditions are suitable for galvanized, polymer coated and ALT2 (DSL = 50 years).

1. Galvanized CSP	16 gage:	EMSL = average 52 years
2. ALT2 CSP	16 gage:	EMSL = minimum 75 years
3. Polymer Coated CSP	16 gage:	EMSL = minimum 100 years

There are three CSP products suitable for this site: 16 gage galvanized, 16 gage ALT2 and 16 gage polymer coated.

Example No. 2

Site Conditions: pH = 5.0, resistivity = 2000 ohm-cm, abrasion = level 2
Water hardness = 125 ppm ($CaCO_3$)
Design Service Life = 70 years
Initial Pipe Selection (structural calculation): 60 in. diameter, 16 gage (5 x 1 in. corrugation)

With r = 2000 ohm-cm, pH = 5.0, EMSL of galvanized pipe (Fig. 91) = 25 years (16 gage)
16 ga. galvanized CSP does not meet DSL and requires a heavier gage or alternative coating.

For 12 gage: EMSL = 1.8 x 25 = 45 years (<70 years so does not satisfy the service life). Use a heavier gage.

For 8 gage: EMSL = 2.8 x 25 = 70 years (OK)

Alternative Pipe Selections:

See Table 9.1 for EMSL of polymer coated and ALT2.
Site environmental conditions are suitable for galvanized, polymer and ALT2 (DSL = 70 years)

1. Galvanized CSP	8 gage:	EMSL = average 70 years
2. ALT2 CSP	16 gage:	EMSL = minimum 75 years
3. Polymer Coated CSP	16 gage:	EMSL = minimum 100 years

Three cost-effective CSP products are suitable for this site: 8 gage galvanized, 16 gage polymer coated and 16 gage ALT2).

Example No. 3

Site Conditions: pH = 4.0, resistivity = 1000 ohm-cm, abrasion level = 3
Water hardness = 50 ppm ($CaCO_3$)
Design Service Life = 75 years
Initial Pipe Selection (structural calculation): 54 in. diameter, 14 gage (2 2/3 x 1/2 in. corrugation)

Alternative Pipe Selections:

1. Galvanized pipe is not recommended at this site because pH, resistivity, and water hardness are outside recommended environmental ranges for this coating. See Table 9.1.
2. ALT2 is not recommended at this site because the pH is below 5.0, the resistivity is below 1,500 ohm-cm and the abrasion level is greater than 2.
3. Polymer coated CSP will perform well in all the site environmental conditions with a minimum service life of 75 years.

The best pipe for installation at this site is 14 gage polymer coated. This pipe will perform well at this site, where the pH is too low even for concrete pipe.

SPECIAL PIPE AND APPLICATIONS

Spiral Rib Pipe

The previous design examples focused on traditional CSP corrugation profiles such as 2 2/3 x 1/2 inch and 5 x 1 inch corrugation. CSP fabricators offer a range of pipe wall corrugations to help the designer meet both hydraulic and structural conditions at each site. An alternative product developed over the last few decades is spiral rib pipe in which the corrugation consists of a smooth interior wall with a helical rectangular rib projecting to the pipe exterior at a spacing of 7.5 or 11.5 inches. The advantage of this wall corrugation is that it provides a CSP product with hydraulic performance equal to that of reinforced concrete pipe and HDPE pipe. This is an economic advantage because it is not necessary to use a larger sized CSP to achieve hydraulic performance equal to that of other pipe materials. Because of the smooth interior, spiral rib pipe is less susceptible to coating loss due to abrasion than traditional CSP corrugations.

Storm Sewers

Most of this chapter has dealt with culvert applications. CSP is also widely used in storm sewer applications. The durability evaluation of storm sewers differs greatly from that of culverts. The major difference is that storm sewers rarely experience high velocities or bed loads of any significance. Flow through storm sewers tend to be much more intermittent than culverts and storm sewers are usually installed at the minimum slopes required to ensure the storm sewer system remains in a self-cleaning condition. These conditions reduce the impacts of abrasion and corrosion. If the effluent through the sewer system is expected to be soft water, caution should be exercised in the use of galvanized CSP. In the absence of soft water, any CSP that satisfies hydraulic and structural requirements can be used in storm sewers without significant durability concerns. Economics will likely be the deciding factor in evaluating alternatives.

Spiral rib pipe is often used in storm sewers to minimize the size of pipe required and thereby reduce the depths at which the downstream lines must be placed. Another distinct advantage of using spiral rib pipe is that it enables the designer to specify three alternative pipe materials (CSP, HDPE and concrete), all of which will require the same size pipe. The smooth nature of the inside of the spiral rib pipe further reduces the already minimal effects of abrasion on the pipe wall. With little or no abrasion forces to damage the interior pipe wall, the EMSL of CSP, when used in storm sewer applications, can be expected to be much greater than when used in culvert applications. A study conducted by the Federal Lands Highway Division of FHWA concluded that 16 gage galvanized CSP would have a service life 25 percent longer than predicted when using the CALTRANS chart. This conclusion was based on their interpretation of the data from their study of sites where abrasion conditions were classified as FHWA Levels 1 or 2.

Given the less aggressive nature of the effluent flowing through storm sewers, the site conditions of the pipe exterior may be the determining factor in calculating pipe service life. A study of the soil side durability of sewer pipe, conducted by the Corrpro companies, and reported in 1991, concluded that where soil side conditions controlled, more than 90 percent of galvanized coated CSP installations would have a service life in excess of 75 years and more than 80 percent would have a life in excess of 100 years. These service life levels should certainly exceed the design service life for the vast majority of storm sewer installations.

Underground Stormwater Systems

CSP is the most widely used material in the construction of underground stormwater storage systems. These systems are used to capture and hold stormwater during storms and then release the stored stormwater at a controlled rate established by local regulations. These systems vary in size from 24 inch diameter to the maximum size of 144 inch diameter. An integral aspect of these systems is the use of many fittings, bulkheads, access manholes, weirs, and bypass piping. Simply stated, these are stormwater storage containers. Stormwater flows into the systems, is temporarily stored there, and is gradually discharged into the storm sewer system at a rate approved by the local regulators.

The flows into these systems are consistent with those generally encountered in storm sewer systems. This means inlet and outlet velocities are minimal and are too low to create abrasive forces within the system.. The flow rate of effluent from the systems is usually less than the inflow rate. These systems essentially operate without developing abrasive conditions. As discussed in the previous section on storm sewers, the CSP in these systems will provide an EMSL significantly greater than the DSL.

NCSPA commissioned a study to evaluate the condition of 17 stormwater storage systems that had been in place for as long as 25 years. The majority of the systems were galvanized coated pipe with no supplemental coatings. Parsons Brinkerhoff and Corrpro performed the visual inspection and evaluated coupons from the pipe walls. The conclusion of the evaluations was that the systems had experienced little or no deterioration over many years of operation, and would have a service life significantly in excess of 100 years.

Steel Structural Plate

Steel structural plate (SSP) has been in use for over 70 years and has a proven performance record. SSP is used where the site requires a structure rather than a manufactured pipe due to the size of the required opening or the depth of earth cover over the structure. SSP is used where the structure span or pipe diameter is larger than is available in CSP. To accommodate the larger sized structure, individual plates are shipped in small sizes and bolted together on site. For a number of reasons, durability is less of a concern with structural plate than with CSP. The thickness of the structural plates is greater than the typical wall thicknesses used in CSP, and the thickness of the plates that form the invert of the structure can be made with steel plates heavier than used in the balance of the structure in order to enhance durability. The maximum thickness of SSP plates is 0.380 inches, nearly 6 times the thickness of the 16 gage CSP used as the basis for Figure

9.1 (EMSL). CSP typically has a galvanized coating weight of 2 oz/sq. ft (total both sides), while SSP is coated after fabrication with a galvanized coating that is 50 percent heavier at 3 oz./sq. ft. With the increased steel thickness and the 50 percent heavier coating thickness, SSP can easily have a service life as much as 8 or 9 times that of 16 gage CSP.

Another advantage of structural plate is that some of the shapes, such as arches and box culverts, can be assembled without an invert. Having no invert eliminates the portion of the pipe or structure which is of greatest concern regarding durability. Another method used to create natural inverts in a structure or pipe is to bury the structure deeper than normal and fill the pipe interior with natural fill material up to the stream gradient. An alternate means of increasing service life of SPP at a given site is to specify an asphalt or concrete paving in the invert of the pipe.

ADDITIONAL INFORMATION

For more specific information on available coatings, linings, and pavings, consult with your local CSP fabricators. Local fabricators can be located on the NCSPA website at www.ncspa.org. Their knowledge and experience will be of great value when dealing with issues related to the service life of CSP products. As can be seen from the example designs, the evaluation of durability is not an exact science. However, over 100 years of experience provides CSP suppliers with a great deal of practical knowledge. Using this knowledge it is possible to make reasonable estimates of durability given basic information related to environmental conditions at the planned installation site. Take advantage of this experience.

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Typical CSP underground detention system.

Proper backfill and equipment are important for a successful installation.

INTRODUCTION

This chapter presents information of fundamental importance regarding installation and construction procedures including base preparation, unloading, assembly, and placement and compaction of the backfill (see Figure 10.1). Procedures for both shop fabricated corrugated steel pipe and field assembled structural plate structures are provided. The emphasis is on corrugated steel pipe in installations such as highway culverts and storm drain pipe. For additional information, reference may also be made to the NCSPA "Installation Manual" (Figure 10.2), and to ASTM recommended practices A 796 / A 796 M, A 798 / A 798 M, and A 807 / A 807 M. The National Corrugated Steel Pipe Association's member manufacturers are also an excellent source for local installation requirements and recommendations.

Figure 10.1 Proper compaction equipment close to structure.

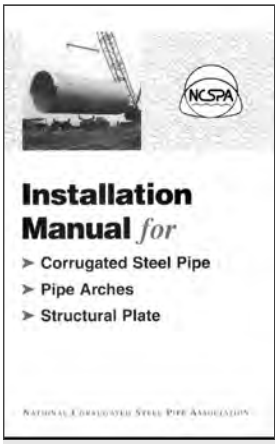


Figure 10.2 NCSPA Installation Manual

A well situated, properly bedded, correctly assembled, and carefully backfilled steel drainage structure will function properly and efficiently over its design life. Although smaller structures and structures with low cover may demand less care in installation than larger ones, reasonable precautions in handling, base preparation, assembly and backfilling are required for pipes of any material.

Corrugated steel structures, because of their strength, light weight and resistance to fracture, can be installed quickly, easily and with the least expensive equipment. The flexible steel shell is designed to distribute external loads to the backfill around it and function as a soil-steel structure interaction system as shown in Figure 10.3. Such flexibility permits unequaled tolerance to settlement and dimensional changes that would cause failure in rigid structures and other types of flexible pipe. This clear advantage of corrugated steel structures is further strengthened when they are installed on a well prepared foundation, and surrounded by a well compacted backfill of stable material. Reasonable care during installation is required. Just as with drainage structures of concrete or other materials, careless installation of corrugated steel structures can undo the work of the designer.

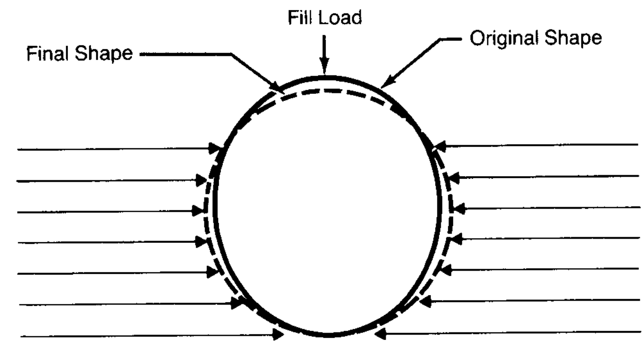


Figure 10.3 Corrugated steel pipe functions structurally as a flexible ring that is supported by and interacts with the compacted surrounding soil. The soil constructed around the pipe is thus an integral part of the structural system. Therefore it is important to ensure that the soil structure or backfill is made up of acceptable material and that it is well-constructed.

In Chapter 7, Structural Design, minimum cover requirements were presented for corrugated steel pipe under highway and railway loadings. These requirements are based on years of practical experience, as well as fundamental design criteria. However, it must be emphasized that such minimum covers may not be adequate during the construction phase, because of the higher live loads incurred. Therefore, when construction equipment that produces wheel loads or gross loads greater than those for which the pipe has been designed, is to be driven over or close to the structure, it is the responsibility of the contractor to provide any additional cover needed to avoid damage to the pipe. More information regarding construction load requirements is found under the headings, *Important Considerations During Compaction,* found later in this chapter.

BASE PREPARATION

Foundation

Pressure developed by the weight of the backfill and live loads is transmitted both to the side fill and the strata underlying the pipe. The supporting soil beneath the pipe, generally referred to as the foundation (Figure 10.4), must provide both longitudinal and lateral support.

A properly developed foundation will:

- Maintain the conduit on a uniform grade.
- Aid in maintaining desired cross-sectional shape.
- Allow for uniform distribution of loading without developing stress concentrations in the pipe wall.

Figure 10.4 A properly developed foundation is important.

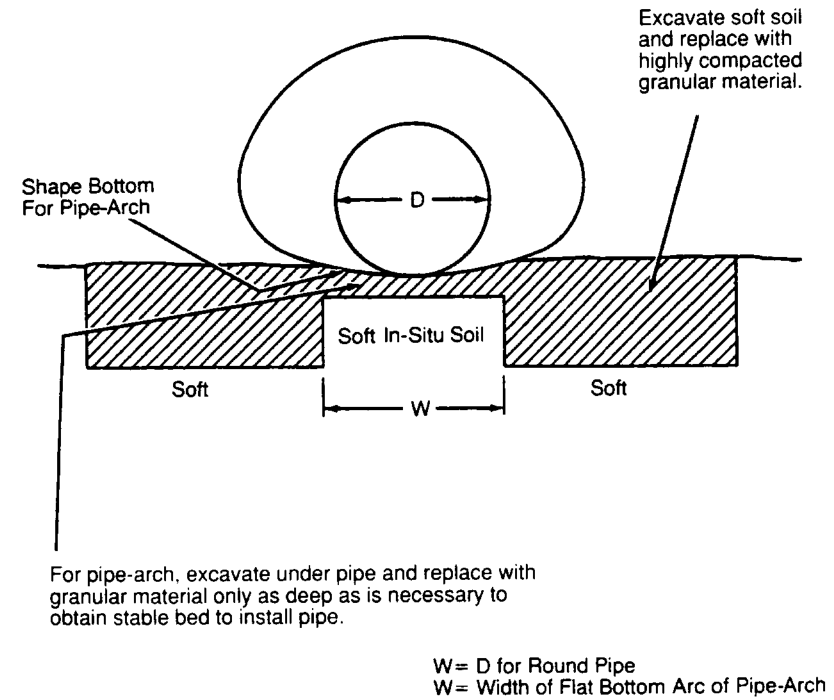
Preliminary Foundation Considerations

Soft Foundation

Evaluation of the construction site may require subsurface exploration to detect undesirable foundation material, such as soft material (muck) or rock ledges. Zones of soft material give uneven support and can cause the pipe to shift and settle non-uniformly after the pipeline is constructed. Materials with poor or non-uniform bearing capacity should be removed and replaced with suitable compacted fill to provide a continuous foundation that uniformly supports the imposed pressures. The bedding may then be prepared as for normal foundations. Figure 10.5 illustrates the treatment of soft foundations.

It is important that poor foundation material be removed for a suitable distance on either side of the pipe and replaced with compacted backfill. Otherwise, that material will settle under the load of the backfill alone and actually increase the load on the pipe. This is referred to as “dragdown soil loading”.

Reference the Structural Design Chapter for further information regarding the treatment of soft foundations.



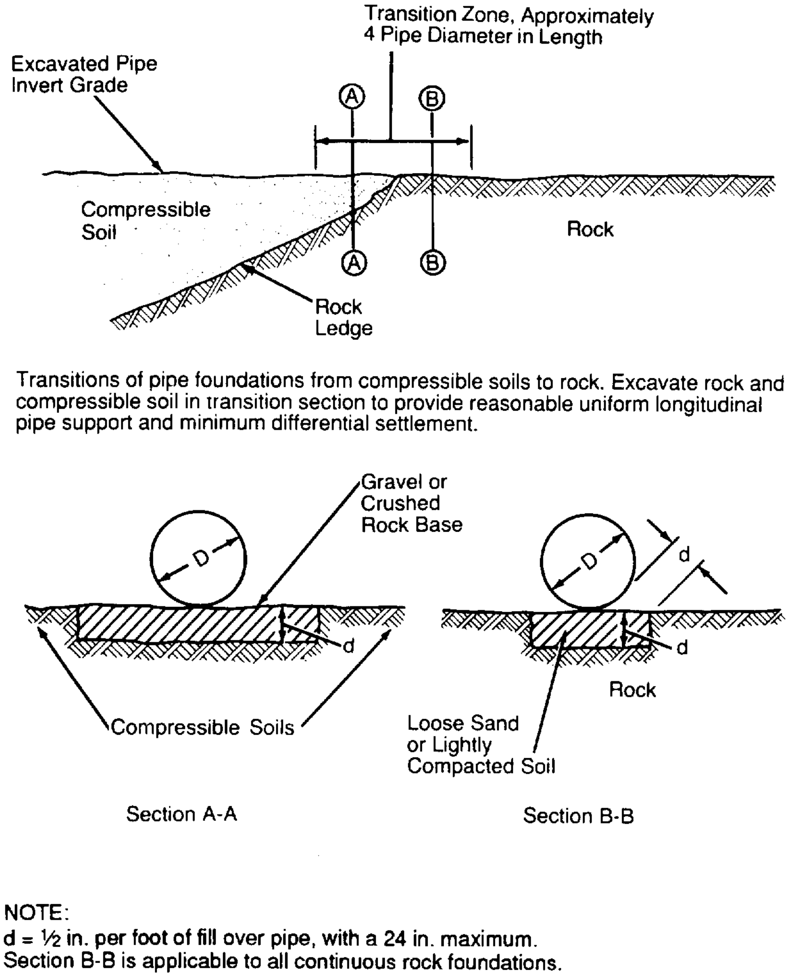
■ **Figure 10.5** Soft foundation treatment.

Rock Foundations

If rock ledges are encountered in the foundation, they may serve as hard points that tend to concentrate the loads on the pipe. Such load concentrations are undesirable because they can lead to distortion of the structure. Thus large rocks or ledges must be removed and replaced with suitable compacted fill before preparing the pipe bedding. Furthermore, when the pipe foundation makes a transition from rock to compressible soil, special care must be taken to provide for reasonably uniform longitudinal support. Figure 10.6 illustrates the treatment for rock foundations and transition zones.

Bedding Foundation

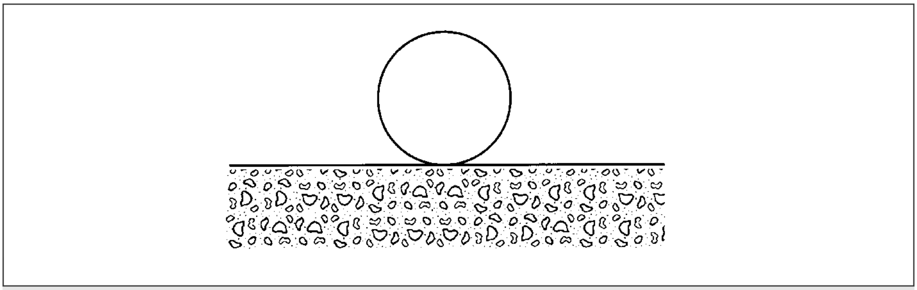
The portion of the foundation in contact with the bottom of the structure is referred to as the bedding. Depending upon the size and type of structure, the bedding may either be flat or shaped. Good bedding foundations can be viewed as a "cushion" for the conduit and should be relatively yielding when compared with compacted material placed between the trench wall or embankment and the pipe. In this manner, a soil arch can develop over the pipe, thus reducing the load transmitted to the conduit.



■ **Figure 10.6** Rock foundations and transition zones.

Normal Round Pipe Bedding

With flat bedding (Figure 10.7), which is typical for factory-made round pipe, the pipe is placed directly on the fine-graded upper portion of the foundation. Soil must then be compacted under the haunches of the structure in the first stages of backfill.



■ **Figure 10.7** Pipe placed on flat bedding.

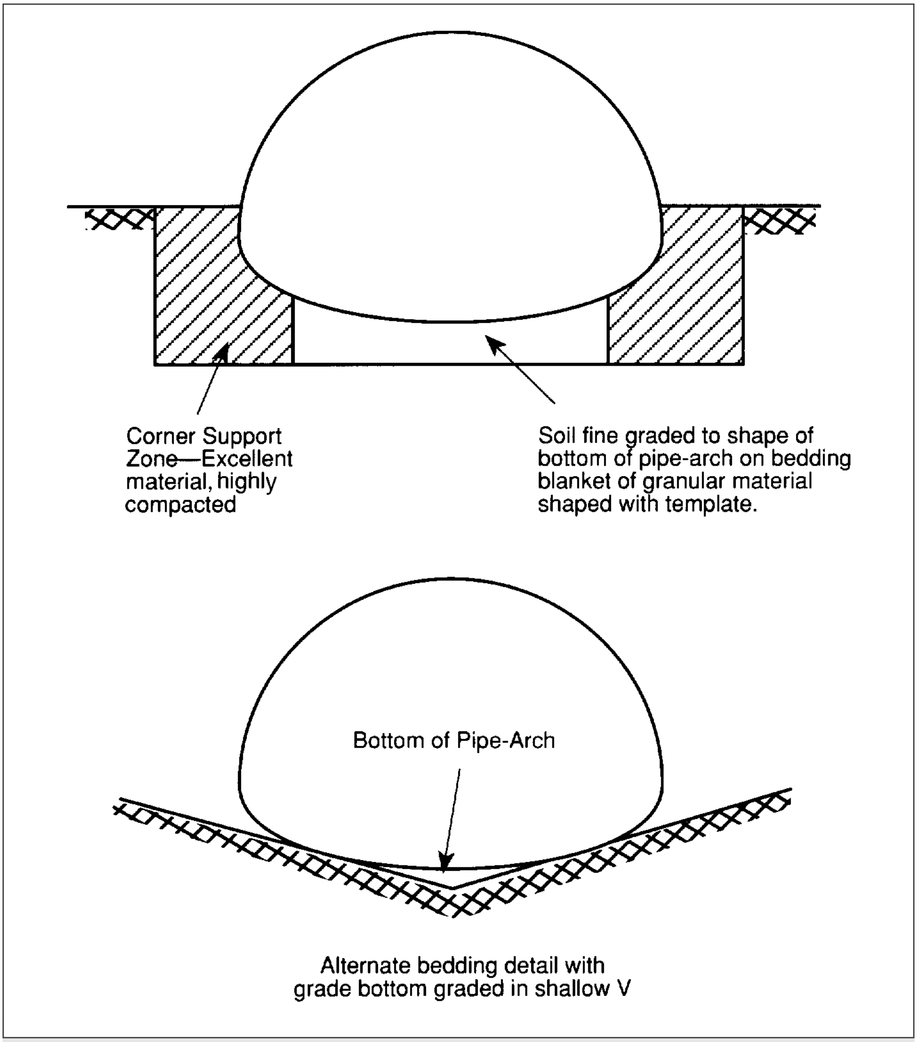
The upper 2 to 6 inch layer should be relatively loose material to allow the corrugations to seat in the bedding. The material in contact with the pipe should not contain gravel larger than 3 inches, frozen lumps, chunks of highly plastic clay, organic matter, or other deleterious material. Figure 10.8 shows bedding for parallel runs of pipe.

■ **Figure 10.8** Proper bedding is important for multiple barrel installations.

Special Bedding Considerations

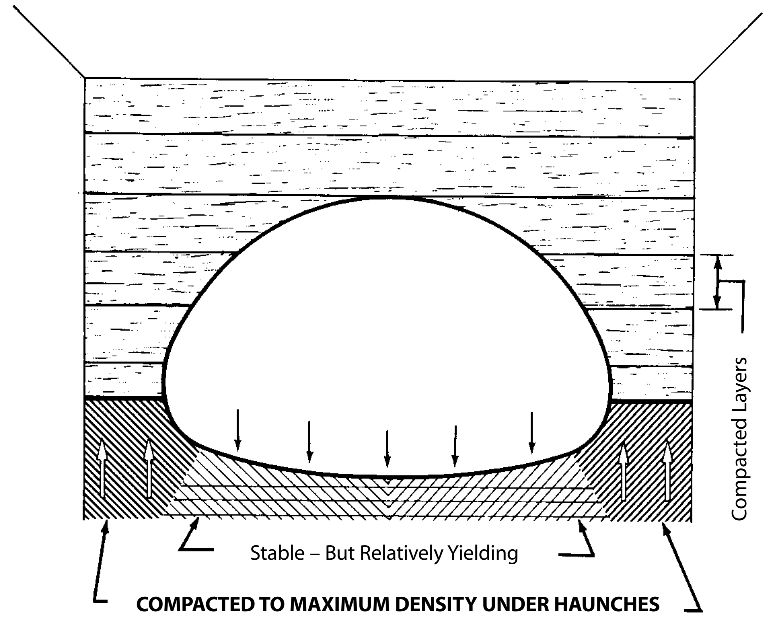
Pipe Arch and Large Diameter Pipe

The bedding concept for pipe arch structures also relates to large diameter and underpass shapes. For these structures, the bedding should be shaped to the approximate contour of the bottom portion of the structure. Alternatively, the bedding can be graded to a slight V-shape. Shaping the bedding affords a more uniform support for the relatively flat bottom of these structures. The shaped portion need not extend across the entire bottom, but must be wide enough to permit the efficient compaction of the backfill under the remaining haunches of the structure.



■ **Figure 10.9** Shaped bedding for pipe arch and large diameter structures.

Figure 10.9 illustrates shaped bedding for a pipe arch. Note that the soil beside and below the corners of a pipe arch must be of excellent quality, highly compacted, and thick enough to spread and accommodate the high reaction pressures that can develop at those locations. It is important in pipe arch installation to ensure a favorable relative movement of the haunches with respect to the pipe bottom. For this reason, a slightly yielding foundation under the bottom, as compared to the haunches, is desirable. This factor is illustrated in Figure 10.10.



■ **Figure 10.10** Pipe arch loads are carried at the corners. Arrows show the direction of favorable relative motion of all pipe arches.

Submerged Bedding

Preferably, the bedding and backfill operation should be conducted entirely in the dry. In rare cases, however, the installation of corrugated steel pipe may have to be done "in-the-wet". For sites where it is not possible or practical to divert the stream, it is common practice to pre-assemble and lift, roll, or skid CSP or Structural Plate Pipe into place. Since such conditions make it very difficult to ensure good base preparation and proper backfill compaction, the designer should consider quality granular backfill materials that achieve the required strength when dumped into standing water. Expert advice is recommended.

Camber

For embankment installations, camber in the grade under high fills, or on a foundation that may settle, should be considered in base preparation. Camber is simply an increase in the foundation or bedding elevation at the center of a culvert above a straight line connecting its ends (the intended grade or slope of the pipe). The objective is to shape and/or elevate the grade to assure a proper flow line after settlement takes place. This forethought will prevent sag in the middle of the culvert that might pocket water, or reduce hydraulic capacity because of sedimentation. Generally, enough camber can be obtained by placing the base for the upstream half of the pipe on an almost flat grade, and the downstream half on a steeper than normal grade. The greater load at the center of the embankment and the corresponding settlement will result in the desired positive slope after full consolidation. Soils engineering techniques are available to predict the amount of camber required for unusual conditions. It is possible to obtain a camber in the structure equal to one-half of one percent of its length without special fittings. For structures under high fills, the ordinates of the camber curve should be determined by a soils engineer. Figure 10.11 illustrates camber for a pipe under a high fill.

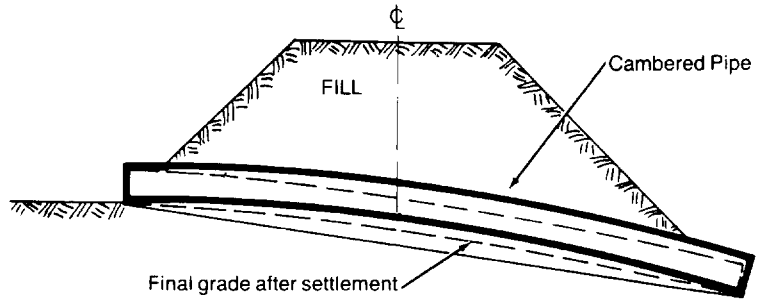


Figure 10.11 Camber allows for settlement of a culvert under a high fill. Most of the fall is in the outlet half. Diameters 10 feet and smaller are easier to camber, as are the lighter wall thicknesses.

INSTALLATION OF CORRUGATED STEEL PIPE AND PIPE ARCH

Unloading and Handling

Although corrugated steel drainage structures will withstand rough handling without deformation, they should be handled with reasonable care. Pipe should never be dumped directly from a truck bed while unloading, but should be lifted or rolled to protect the coated surface. Dragging the structures at any time may damage the coatings and jeopardize durability. Also, avoid striking rocks or hard objects when lowering pipe into trenches.

Corrugated steel structures are relatively light in weight and can be handled with simple, light equipment. If necessary, a small crew can lower pipe into trenches by means of rope slings. Where the pipe is to be set in a trench, it is necessary to have equipment with a large enough reach to allow proper rigging of the lifting straps, cables, etc. as seen in Figure 10.12.

■ **Figure 10.12** Lifting CSP into place.

Lifting Lug Locations

The recommended technique for lifting sections of pipe is the use of slings whenever possible. In situations where the use of slings is not possible, lifting lugs can be used. Figure 10.12a shows a diagram of the recommended lifting lug location for pipe loads up to 4,000 lbs utilizing 2 lugs. Figure 10.12b covers 4 lugs and loads up to 8,000 lbs.

■ **Figure 10.12a** Lifting lugs for pipes to 4000 lb.

■ **Figure 10.12b** Lifting lugs for pipes to 8000 lb.

Assembly

Coupling

The usual method of joining two or more lengths of pipe or pipe arch is by means of steel connection bands. The bands engage the ends of each pipe section and are placed to overlap an equal length of each pipe providing an integral and continuous structure. During the construction of a corrugated steel pipe system, care should be given to the treatment of joints to prevent both infiltration and exfiltration. Both processes will have an adverse effect upon backfill materials, since soil particle migration can occur. This is particularly true when fine-grained soils (silt and clay) are present in the backfill material. The addition of a geosynthetic wrap or gasket material around the pipe joint can provide additional soil tightness to the coupling system. See Figure 10.13 below.

Figure 10.13 CSP pipe joints wrapped with geosynthetic wrap.

Performance requirements are published in Division II, Section 26.4.2, of the current edition of the AASHTO Bridge Specifications. The AASHTO Specifications provide an excellent description of the different joint types and properties. Joint properties include shear strength, moment strength, tensile(pull-apart) strength, joint overlap, soil tightness and water tightness. Their recommended minimum requirements depend on whether the pipe is being installed in erodible or non-erodible soil. It should be emphasized that most corrugated steel pipe installations will only require a standard joint.

Typical Corrugated Steel Pipe Band Installation

One-piece bands are used on small diameter pipe. Two or three-piece bands are used on larger diameter pipe (see Figure 10.14) and when installation conditions are difficult. Rod and lug bands are used on levees, aerial sewers and similar installations where improved water-tightness (or beam strength) is essential. Bands utilizing gaskets are commonly used in restricted leakage applications. Specially fabricated connectors can be supplied for use in jacking and for special or unusual conditions.

Figure 10.14 Pre-fabricated miter section of CSP is lowered into place to match bend in trench.

Bands are put into position at the end of one section of pipe with the band open to receive the next section, depending on the type of band, the second section is brought against or to within about 1 inch of the first section as seen in Figure 10.15. After checking to see that connecting parts of both band and pipe section match, and that the interior of bands and exterior of pipe are clean, bolts are inserted and tightened.

To speed the coupling operation, especially for large diameter structures, a chain or cable-cinching tool will help tighten the band. Special clamping tools are available that fit over coupling band connectors and quickly draw the band together. Such devices permit faster hand tightening of the bolts, so that a wrench is required only for final tightening.

On large diameter structures, merely tightening the bolts will not assure a tight joint because of the friction between the band and the pipe ends. In such installations, tap the band with a rubber mallet to cause it to move relative to the pipe as the band is tightened. The wrench used to tighten coupling bands can be a box end wrench, but greater assem-

■ **Figure 10.15** Band used to join reformed end helical pipe.

bly speed can be accomplished with a speed wrench or ratchet wrench equipped with a deep socket.

More information is available on the different band types and assembly instructions in the NCSPA "Installation Manual".

Coated Pipe Band Installation

On coated pipe, the surface between coupler and pipe may need lubrication with vegetable oil or a soap solution. This will allow the band to slip around the pipe more easily and to draw it into place more firmly, particularly in cold weather. Lubricating and tapping the band, with a rubber mallet, as it is tightened will help ensure a good joint. Where damage to the coating exposes the metal, repair by patching should be done before the structure is backfilled. A suitable repair material is asphalt mastic.

Paved Invert Pipe Band Installation

Pipe with bituminous pavement must be installed with the smooth, thick pavement in the bottom. To simplify such placement and to speed handling, paved invert pipe lengths may be ordered with metal tabs or lifting lugs fastened to the pipe exterior exactly opposite the location of the pavement. Slings, with lifting hooks inserted in the lugs, automatically locate the paved invert in the bottom of the structure. Band installation is similar to that described above.

Pre-Fabricated Pipe Fittings and Field Adjustments

Manholes

Shop fabricated corrugated steel manholes are available for all shapes of corrugated steel pipe structures. They are designed to receive standard cast iron appurtenances such as manhole covers and grates. Corrugated steel manholes have the advantage of quick installation and backfilling, thus reducing the possibility of damage to the pipeline due to flooding caused by unexpected weather conditions. Installation of a manhole riser is seen in Figure 10.16.

Figure 10.16 CSP manhole being installed.

Manholes are multipurpose in function. They provide access for maintenance, serve as junction chambers where several conduits are joined together, provide an inlet for storm water from a grate inlet and are used to facilitate a change in horizontal or vertical alignment.

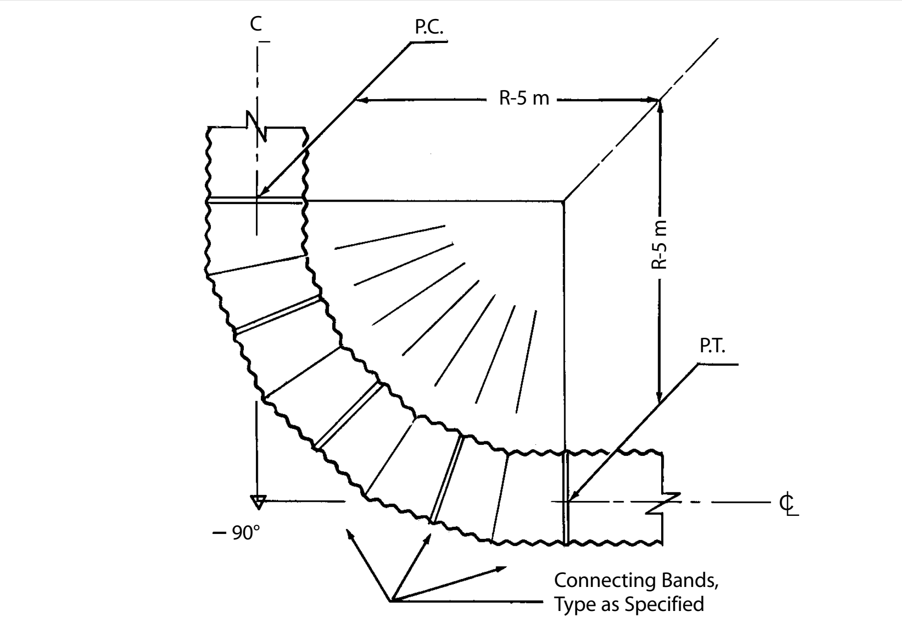
Monolithic concrete junction chambers are usually square or rectangular in shape. Structures of this design have the distinct disadvantage of causing turbulent flow conditions that, in effect, reduce the carrying capacity in upstream portions of the conduit system.

It is frequently desirable to change the horizontal or vertical alignment of large diameter corrugated steel drainage structures without the use of a manhole or junction chamber.

Shop fabricated elbow sections are available for this purpose and, in most instances; the additional fabrication cost is more than offset by eliminating the manhole or junction chamber. Manhole design is discussed in Chapter 8.

Elbows, Tees and Wyes

Elbow pipe sections can be prepared to provide gradual changes in flow direction. Such fittings are prepared from standard pipe, pipe arch or arch sections and have the advantage of providing a change in direction without interrupting the flowline. Figure 10.17 graphically indicates the form of these sections that are available in any increment between 0° and 90°. Elbow fittings can be used in conjunction with each other, thus providing a custom design to accommodate required field conditions. For example, a horizontal alignment change of 90° could be negotiated through the use of three 30° or four 22 1/2 ° sections. A horizontal shift in alignment can easily be accommodated by the use of two elbow fittings, with the second fitting simply installed in reverse orientation to the first.



■ **Figure 10.17** Alignment for pipe elbow sections. The above is a design to negotiate a 90° alignment change through the use of four 22 1/2° sections.

The use of special fittings and elbow sections requires precise surveys both in the design and layout stages. The accurate location of special items must be predetermined in order for the manufacturer to supply fittings and straight pipe sections that will conform to field conditions. Layout and installation must be done with care to ensure proper positioning of all portions of the corrugated steel pipe system. The field layout procedure for elbow pipe sections involves geometry similar to that of a standard highway curve. It should be noted, however, that only the center points at the end of each elbow section lie on the path of the circular curve.

Saddle Fittings

Saddle fittings are available to aid the connection of laterals or other branches entering or leaving the main structure. Figure 10.18 demonstrates an example of a saddle fitting. They are especially useful where the exact location or grade of existing tie-ins are unknown prior to construction. While the longitudinal location of a saddle fitting must be spaced to the pitch of the corrugation, any line at any angle may be joined to the main or line simply by cutting or sawing the required hole. The saddle branch is attached over this opening and the incoming line is then attached to the fitting.

Figure 10.18 Typical CSP saddle fitting.

PLACEMENT AND COMPACTION OF BACKFILL

Selection of Structural Backfill

The Structural Design Chapter details the importance of backfill selection, specifically pertaining to requirements for large diameter pipes and higher heights of cover, reference to that chapter should be made for those situations.

For the roadway conduit to support the pavement or track above it adequately and uniformly, a stable composite structure is vital. Stability in a soil-steel structure interaction system requires not only adequate design of the structure barrel, but also a well-engineered backfill. Performance of the flexible conduit in retaining its shape and structural integrity depends greatly on the selection, placement and compaction of the envelope of earth surrounding the structure, which distributes its pressures to the surrounding soil masses.

Requirements for selecting and placing backfill material around or near the conduit are similar, in some respects, to those for a roadway embankment. However, a difference in requirements arises because the conduit may generate more lateral pressure than would the earth within the embankment if no structure existed. Therefore, the soil adjacent to the conduit must be well compacted. Standard compaction specifications call for achieving a minimum of 90% standard proctor density (per AASHTO T-99).

Soil Design for CSP

Highway and railroad engineering departments have detailed specifications for selecting and placing material in embankments. These specifications provide for wide variations in terrain and for available local materials. They can generally apply to backfill material around conduits for normal installations. If abnormal conditions exist at a specific site or if unusual performance is expected of a conduit and embankment, a soils engineer should be consulted for designing the backfill.

Backfill material should preferably be granular to provide good structural performance and ease of compaction. Bank, pit run gravel, or coarse sands are usually satisfactory. Very fine granular backfill material may infiltrate into the structure and should be avoided, particularly when a high ground water table is anticipated. When a coarse granular backfill is placed next to a fine native or embankment material, the soils must be separated by a suitable transition soil or filter fabric to control migration into the backfill. Where infiltration is desirable to lower the ground water table, geotextiles are also used to provide the necessary separation function.

Cohesive Backfill

Clay soils are generally not recommended for use as structural backfill. Good compaction of clay soil is difficult to obtain due to the very narrow optimum range for moisture content versus density. It is difficult to maintain allowable moisture content throughout the backfill operation as a result of snow, rain or normal drying. Dry clays need to be broken up or pulverized before placement and brought to the optimal moisture content before compaction. Clays above their optimum moisture content require either a drying operation or time for each lift to air-dry before it is compacted. Generally, shallower lifts are required for acceptable end results.

If clay soils are used, much closer inspection and field testing must be exercised to assure good results. Cohesive material should only be used for small pipes; not for larger structures, and should be limited to lower cover applications. If cohesive backfill material is to be used, geotechnical advice is recommended.

Hydraulic Backfill

Cement slurries, or other materials that set up without compaction, may be practical for unusual field conditions. Limited trench widths or relining of existing structures may warrant the use of self-setting cementitious slurries or grout. Care must be taken to ensure that all voids are filled, and that the material used will provide the compressive strength required. As with water consolidation techniques, measures should be taken to prevent floatation. Some techniques are covered in this chapter under the heading, *flotation*, but, expert advice is recommended.

Backfill and Compaction Density

Experience and research have shown the critical density of backfill to be below 85% Standard Proctor Density. Backfill must be compacted to a greater density than critical to assure good performance. Therefore, backfill for all structures should be compacted to a specified 90% minimum per AASHTO T-99 or greater if required by manufacturers specifications.

Compaction Equipment

Hand Equipment

For compaction under the haunches of a structure, a pole (or 2 by 4 inch), timber, or air tamper is generally needed to work in the smaller areas. Hand tampers for compacting horizontal layers should not weigh less than 20 lbs. and have a tamping face not larger than 6 by 6 inches. Tampers typically used for sidewalk construction are generally too light.

Mechanical Compactors

Most types of power tampers are satisfactory and can be used in all but the most confined areas. However, they must be used carefully and completely over the entire area of each layer to obtain the desired compaction. Care should be exercised to avoid striking the structure with power tamping tools.

Rollers

Where space permits, walk-behind, small riding or rubber-tired rollers, as well as other types of tamping rollers, can be used to compact backfill around the structure. If rollers are used, fill adjacent to the structure should be tamped with hand-held power equipment. Be sure to keep the rollers from hitting the structure. Generally, sheep-foot rollers are used for compacting only clay backfill or embankment material.

Vibrating Compactors

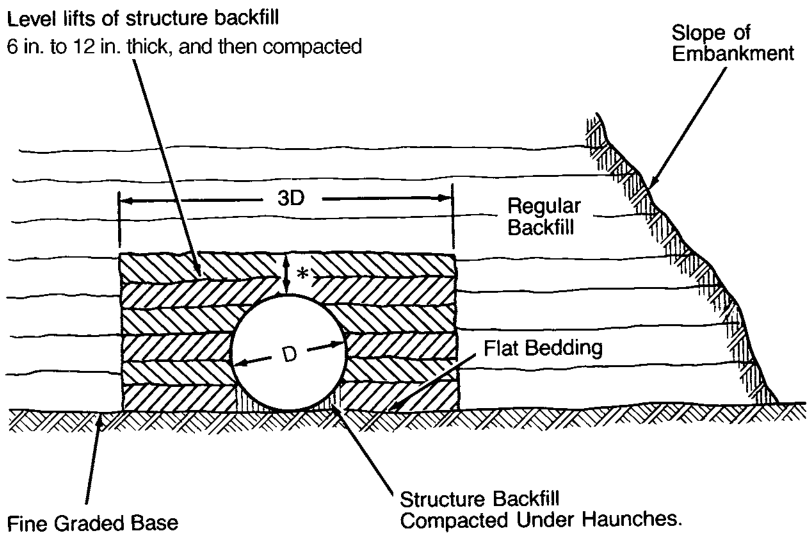
Vibrating equipment is excellent for compaction of granular backfills, but generally is unsatisfactory for clay or other plastic soils.

Flooding and Jetting

Flooding or jetting backfill for consolidation is only effective where the foundation material is able to take the water out of the backfill quickly. The rapid movement of the water carrying the finer backfill material down into the lower levels of the backfill achieves consolidation. Only clean, well graded sand and gravels can be consolidated by this means.

Placing Backfill Around the Structure

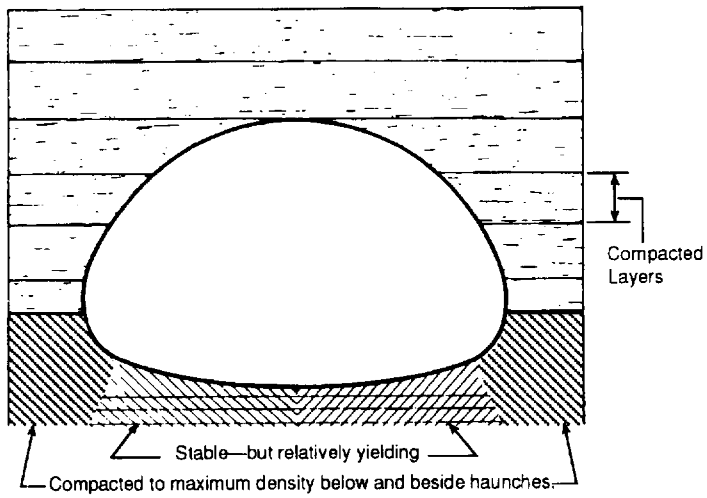
Fill material under haunches and around the structure should be placed in loose layers 6 to 12 inches thick, depending on the quality of the backfill material, to permit thorough compaction. The backfill shall be placed and compacted with care under the haunches of the pipe and shall be raised evenly on both sides of the pipe by working backfill oper-



* Minimum cover of structure backfill for highways is D/8 or 12 in., whichever is greater.

Figure 10.19 Typical backfill envelope for round pipe installed on flat bedding, in an embankment condition.

ations from side to side. The side to side backfill differential shall not exceed 24 inches or one-third of the rise of the structure, whichever is less. Figures 10.19 and 10.20 show how round and pipe arch structures should be backfilled. Pipe arches require that the backfill at the corners (sides) be of the best material, and be especially well compacted.



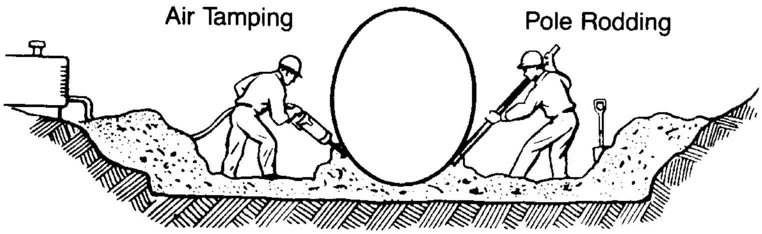
■ **Figure 10.20** Compaction below and beside the haunches of pipe arches is important.

Compaction can be done with hand or mechanical equipment, tamping rollers, or vibrating compactors, depending upon the type of soil and field conditions. Placing the fill material carefully, controlling its moisture content and the lift thickness, will allow for easier compaction of the fill and speed construction.

Steps in Backfill Operation

Backfilling and compacting under the haunches are important steps in the backfill sequence. The material under the haunches must be in firm contact with the entire bottom surface of the structure. The area under the pipe haunches is more difficult to fill and compact and sometimes does not receive adequate attention. Care must be taken to assure that voids and soft spots do not occur under the haunches. Manual placement and compaction must be used to build up the backfill in this area.

Windrow backfill material on each side of the structure and place it under haunches by shovel. Compact firmly by hand with 2 by 4 inch tampers, or suitable power compactors (Figure 10.21). Continue placing backfill equally on each side, in uncompacted layers from 8 to 12 inches in depth, depending on the type of material and compaction equipment or methods used. Each layer must be compacted to the specified density before adding the next. These compacted layers must extend to the trench wall or to compacted embankment material.



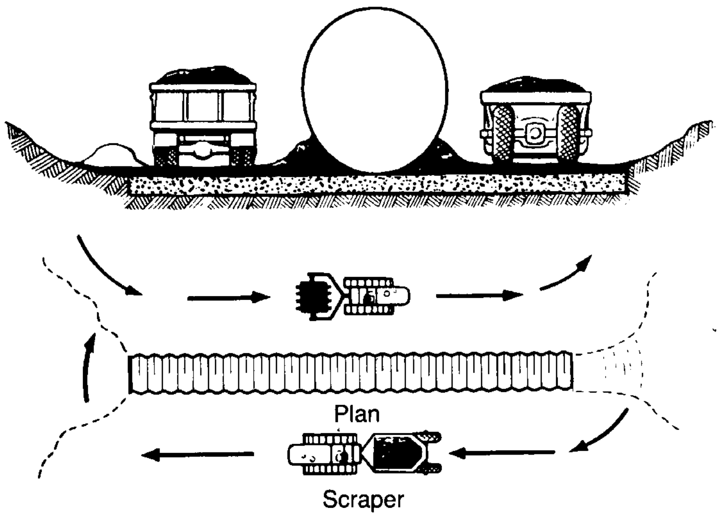
■ **Figure 10.21** Backfill under the haunches should be placed and compacted by the most economical methods available, consistent with providing uniform compaction without soft spots.

Backfill in the corrugation valleys and the area immediately next to the pipe should be compacted by hand-operated methods (Figure 10.22). Heavy compaction equipment may approach as close as 3 to 6 feet, depending on the size of the structure. Any change in dimension or plumb of the structure warns that heavy machines must work further away.

■ **Figure 10.22** Proper use of Compaction Equipment.

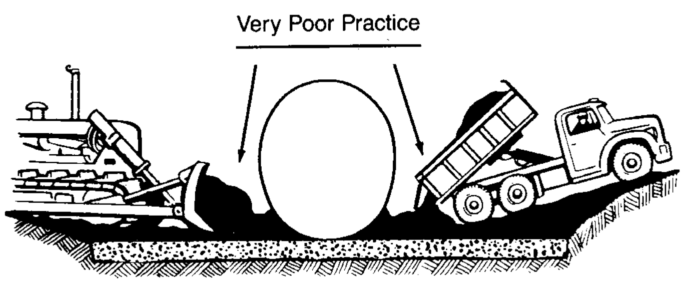
Structural backfill should be compactible soil or granular fill material. Structural backfill may be excavated native material, when suitable. Select materials (not larger than 3 in.), with excellent structural characteristics, are preferred. Desired end results can be obtained with such material with less compaction effort over a wide range of moisture contents, lift thicknesses, and compaction equipment. To ensure that no pockets of uncompacted backfill are left next to the structure and to minimize the impact of material placement and compaction methods, follow this simple rule:

All equipment runs parallel to the length of the pipe (Figure 10.23) **until such time as the elevation of the backfill reaches a point that is at least 3/4 of the rise of the structure.**



■ **Figure 10.23** Good backfilling practice.

Figure 10.24 illustrates poor practices. The possibility of uncompacted fill, or voids next to the structure are bound to arise with equipment operating at right angles to the structure. Mounding and dumping of backfill material against the structure will also adversely effect the installation.



■ **Figure 10.24** Poor backfilling practice.

A balanced sequence of backfilling on either side is recommended:

For embankment installations
- Dump trucks or scrapers windrow granular backfill one-half to one span away (depending on size of structure and site) on either side.
- Graders or dozers spread in shallow lifts for compaction.

For trench installations

Backfill is placed with a loader or stone bucket to a depth to not exceed 3 feet or one third the rise, whichever is less, and then spread to the lift thickness.

For all installations
- Pedestrian-type compactors are used for close work, while heavier self-propelled vibratory drum compactors are used away from the structure and for the rest of the soil envelope, once minimum cover is achieved.
- Supervision of material placement and compaction methods and inspection of pipe shape provide invaluable feedback.
- Hand work, or very light equipment, is used over the top of the structure until minimum cover is achieved.
- Monitor the shape of the structure during backfill. A slight peaking (increase in rise) indicates compaction is being achieved. Pushing it out of plumb means heavy equipment is working too close or the backfill is being placed from one side.

Drainage and Hydraulic Protection During Backfill Operation

During installation (prior to the completion of backfilling, permanent end treatment, slope protection and flow controls) the structure is vulnerable to storm and flow conditions that may be less than the final design levels. Hydraulic flow forces on unprotected ends, unbalanced backfill loads, loss of backfill and support due to erosion and flotation uplift forces, are examples of factors to be considered. While guidance is offered in some of the above sections, temporary protection may be necessary during construction.

Hydraulic forces can float incomplete structures without protection or buckle inverts if the foundation, bedding or backfill becomes inundated. Large radius inverts are especially vulnerable to buckling. If flow is channeled through a structure that is being installed, placing end treatments and slope protection as early as possible are advised. Temporary clay dikes can direct the water flow into the pipe. Protect structures that have coffer dams. Protect trench installations from surface runoff and ponding. Storm sewers and other pipes with inlets need to have branches properly connected so flow is into the main line, not the trench.

In order to provide proper drainage of the backfill above the spring line, it is desirable to grade or slope the fill slightly toward the ends of the structure (where headwalls are not present). This also facilitates fill over the crown, or locking-in the structure. Conversely, if headwalls are built prior to backfilling, work should proceed form the ends towards the middle. Both of these approaches are shown in Figure 10.25. The headwall first approach is useful where it is desirable to divert the stream through the structure and/or to give cut and fill access from both sides at an early stage. Care must be exercised to provide for surface runoff, to prevent ponding or saturation of the backfill from rainfall or snowmelt.

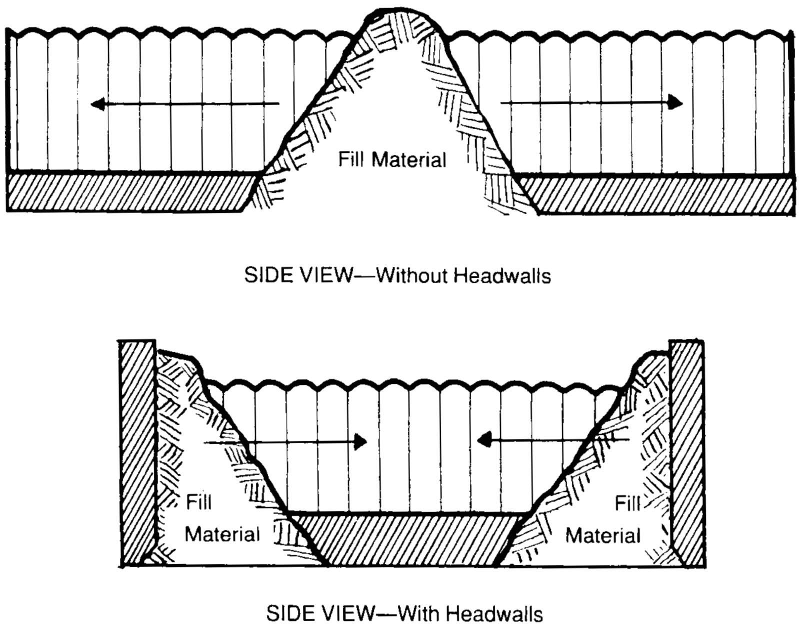


Figure 10.25 Recommended backfilling direction depending on presence of end walls.

Important Considerations During Compaction

Construction Loads

During the construction phase it is sometimes necessary for heavy construction equipment to travel over installed corrugated steel structures during completion of grading, paving or other site work. Heavy construction equipment can impose concentrated loads far in excess of those the structure is designed to carry.

Construction depth-of-cover tables are based on extensive research, as well as experience and fundamental design principles. However, it must be emphasized that the listed minimums may not be adequate during the construction phase because of higher live loads from construction equipment. When construction equipment with heavy wheel loads, greater than those for which the pipe was designed, is to be driven over or close to the structure, it is the responsibility of the installer to provide the additional cover needed to prevent pipe damage. Table 10.1 provides minimum cover guidelines. Steel box culverts are especially vulnerable to damage from excessive live loads and may require additional temporary cover.

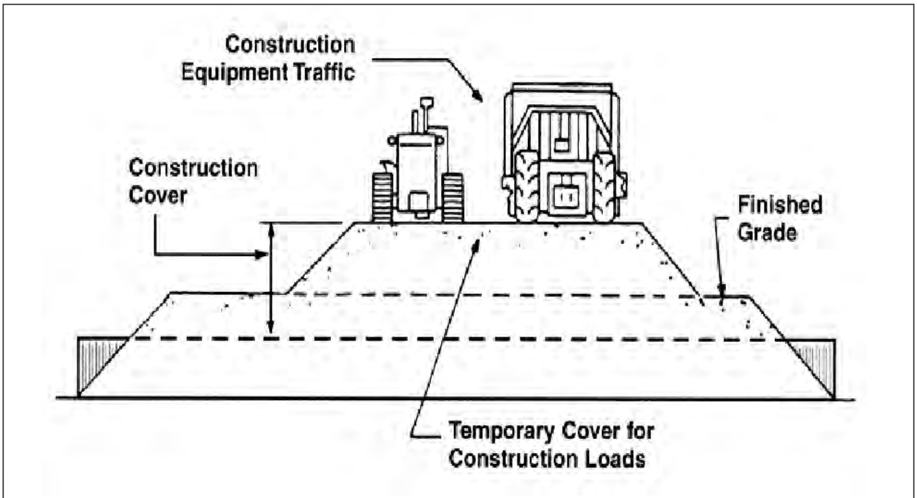
Table 10.1

Guidelines for minimum cover for heavy off-road construction equipment.

Span	Min. Cover (ft) for Axle Loads (kips)			
(in)	18-50	50-75	75-110	110-150
12-42	2.0	2.5	3.0	3.0
48-72	3.0	3.0	3.5	4.0
78-120	3.0	3.5	4.0	4.0
126-144	3.5	4.0	4.5	4.5

1) Min. crossing width of twice the span is recommended.
2) Additional cover may be needed depending on local conditions.

The amount of additional fill needed depends on the equipment axle loads as well as rutting and frequency of use. Figure 10.26 provides safe minimum cover limits for typical structure sizes, axle loads and construction use. This figure does not apply to steel box culverts. The additional temporary cover shown in Figure 10.26 must be maintained so that rutting, surface grading, etc. does not reduce its effect. A minimum crossing width of twice the structure span (or total width for multiple structures) is recommended for typical equipment.



■ **Figure 10.26** Minimum cover for construction loads. (See also Figure 10.42.)

Minimum Cover

When the fill on both sides approaches the top of the structure, the same techniques of spreading shallow layers and compacting thoroughly must be continued as the fill covers the pipe. For the initial layers over the pipe, use of light compaction equipment working across the pipe is recommended.

After minimum cover requirements for the equipment used have been reached, and the structure is locked into place, further filling to grade may continue, using procedures applicable to regular trench or embankment construction.

Shape Control

Shape control refers to controlling the symmetry of the structure during backfill, by control of the backfill operation. Two movements may occur during backfill - "peaking", caused by the pressure of the compacting side-fills, and "rolling", caused by unbalanced fill or greater compaction on one side as shown in Figure 10.27.

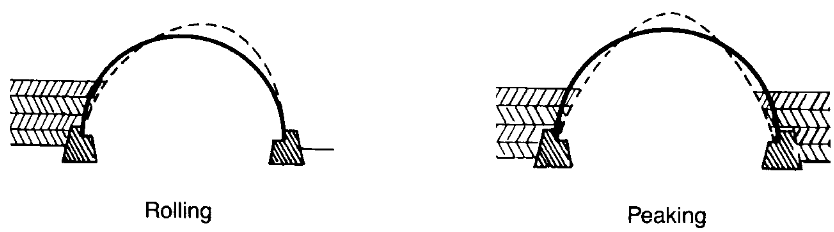


Figure 10.27 Rolling and peaking.

As a general rule, deflection in any direction, measuring greater than 2% from original shape, should not be allowed during the backfill operation. The plumb bob method of deflection monitoring (Figure 10.28) is convenient and effective. Suspend plumb bobs, prior to backfilling, from the shoulder (2 and 10 o'clock) positions so that the points of the bobs are a specific distance above a marked point on the invert. Peaking action can be detected when the points of the bobs move upwards. Corrective action is to keep equipment further away from the structure and/or to be cautious during compaction effort. It is unlikely that peaking will become severe, except for structures with long radius sides (i.e. vertical ellipses, medium and high profile arches, and pear or horse shoe shapes).

Rolling (racking) action can be detected when the plumb bobs move laterally. Early on this is corrected by filling or compacting on the side towards which the plumb bob has moved. For example, a roll to the right will be corrected by placing a higher fill level on the right. Careful monitoring of the plumb bobs and prompt remedial steps prevent excessive peaking or rolling action from distorting the structure.

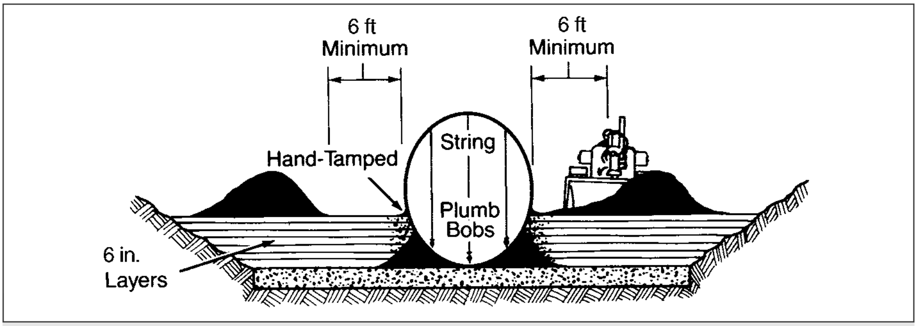


Figure 10.28 Backfilling with plumb bob monitoring.

If distortion greater than what is allowable occurs, backfill should be removed and replaced. The steel structure will usually return to its original shape, unless distortion has been excessive. Shop-cut bevel and skew ends act as cantilever retaining walls and may not be able to resist the lateral pressures caused by heavy equipment and vigorous compaction. Temporary horizontal bracing should be installed across beveled or skewed ends before backfill commences if heavy equipment is to be used close to the cut ends. Alternatively, heavy equipment should be kept away from the cut ends of the pipe. The larger the rise of the structure, the more important this becomes.

Vertical Deflection

The sides of a flexible structure will naturally push outward resulting in compaction of the side-fills and mobilizing their passive resistance. As the sides go outward, the top moves downward (Figure 10.3). This downward vertical deflection is normal. With reasonable backfill practice, any flexible underground structure can be expected to deflect vertically. With excellent practice, the deflection is usually less than 2% of the rise dimension.

If the side-fills are placed loose and/or not compacted, the sides of a flexible structure will move outward to a point where the vertical deflection increases the radius of the pipe crown to the point that pipe failure may occur by buckling. For smaller diameter round pipes, experience has shown that complete vertical (snap-through) buckling failure may occur at 20% to 30% vertical deflection.

Positive soil arching usually occurs over flexible structures with depths of cover greater than the pipe diameter. If the column of fill over the pipe settles slightly more than the side-fills, some of the weight of this column is effectively transferred to the side-fills through shear. In the process, a positive soil arch is mobilized, which reduces the effective load on the structure. Once again, correct installation and backfilling are required for this to occur. The height of cover tables in Chapter 7 assume some soil arching.

Pipe Arch Backfill

Pipe arches require special attention to the backfill material and compaction around the corners. A large proportion of the vertical load over the pipe is transmitted into the soil at the corners (Figure 10.9 in this chapter). The backfill adjacent to pipe arch corners must provide at least 4,000 psf of bearing resistance. In the case of high fills, deep trenches, or soft native soils, a special design may be required for corner backfill zones. Round pipe is recommended in these conditions, rather than the pipe arch shape.

Multiple Barrel Installations

When two or more steel drainage structures are installed in parallel lines, the space between them must be adequate to allow proper backfill placement, haunching and compaction. The minimum spacing requirement depends upon the shape and size of the structure as well as the type of backfill material. Figure 10.29 provides recommended minimum spacing for pipe, pipe arch and arches when standard backfill materials are used. The minimum spacing provides adequate room to fill under the haunches and to compact the backfill.

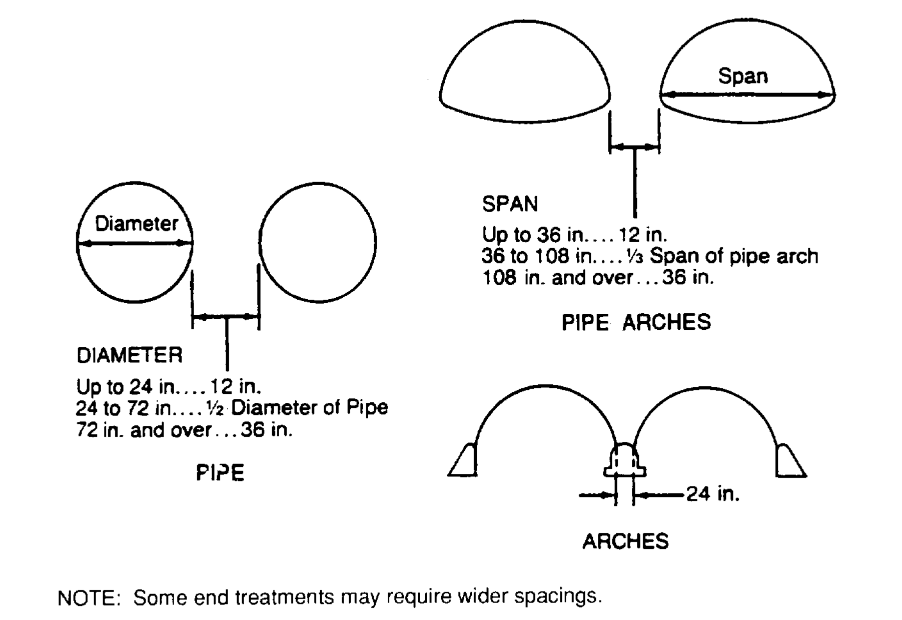


Figure 10.29 Minimum permissible spacings for multiple installations.

Minimum spacing can be reduced somewhat when crushed rock or other backfill materials are used that flow easily (into the haunch) and require little compaction. Spacings of 18 inches or less can be used with backfill materials such as crushed rock, #57 stone or pea gravel. These materials are easier to place in the haunches. When necessary, concrete vibrators can be used to move and consolidate the backfill, much like they do fluid concrete, to assure that there are no voids remaining. When controlled low strength material (CLSM) is used as backfill, the spacing restriction is reduced to the spacing necessary to place the grout between the structures. Regardless of the material, backfilling between and outside the structure cannot be done independently. Rather, backfilling must proceed jointly to maintain a balanced load.

Whether the structures are large or small, the room required for compaction equipment also should be considered in determining spacing between structures. For example, with structural plate structures it may be desirable to utilize mobile equipment for compaction between structures. The space between pipes should allow efficient operation of tamping equipment. Where these limits on structure spacing are cumbersome, use of CLSM between structures often can reduce the spacing requirements to the few inches required for hoses, etc. to place the backfill or the space needed to physically join or assemble the pipes, whichever is greater. There is additional discussion on multiple barrel installations in the detention system section later in this chapter under the heading, Detention, Retention and Recharge Structures.

Floatation

When CSP reline pipes or those backfilled with grout are installed, a primary consideration is the need to control flotation. Fluid grout, which may have a density of 120 pcf or greater can develop greater buoyancy forces than water. To minimize flotation problems, grout is typically placed in thin lifts from side to side of the pipes in a balanced manner.

When it is necessary to place the fluid grout in lifts that produce more buoyant force than the weight of the pipe, the pipe must be held in place. Methods to hold reline pipes down typically include interior bracing against the host structure (see Chapter 12).

Direct burial pipes typically are more difficult to hold down. Techniques that have been used to provide a degree of hold down restraint include placing timbers over the pipe with each end wedged into the trench wall, or placing tension straps over the pipe crown tied to earth anchors in the foundation. Where feasible, pipes have been filled with water or weighted down with concrete blocks placed on roller dollies in the invert. Where the hold down restraints are intermittent, support spacing limits apply such as discussed for aerial spans in Chapter 8. However, it must be recognized that the aerial span limits apply to water filled pipes whereas inundating the entire pipe with grout could develop roughly twice the uplift, due to the higher grout density.

One way to reduce the buoyant forces is the use of CLSM or lightweight cementitious backfill materials. This material is often Portland cement, water and a foaming agent that, at 30 to 40 pounds per cubic foot, provide excellent backfill and lower buoyancy forces than a low strength slurry.

Backfill Summary

In summary, the key points in the backfilling operation are:

- Use good backfill material.
- Ensure good backfill and adequate compaction under haunches.
- Maintain adequate width of backfill.
- Place material in thin, uniform, layers.
- Balance fill on either side of the structure as backfilling progresses.
- Compact each layer before adding the next layer.
- Monitor design shape and modify backfill procedures if required.
- Do not allow heavy equipment over the structure without adequate cover protection.
- Special considerations include multiple radius shapes (pipe arches, underpass, etc.), multiple barrel installations and detention/retention structures.

End Treatment

In many installations, the ends of corrugated steel pipe that project through an embankment can be simply specified as square ends; that is, not beveled or skewed. The culvert length can be increased to accommodate slopes to the bottom of the square end of the pipe. Many times this is the least expensive end treatment. The protection of the soil face should be considered during construction so that erosion is limited. The square end is lowest in cost and readily adaptable to road widening projects. For larger structures, the slope can often be warped around the ends to avoid severe skews or bevels on the pipe end. When desired for hydraulic considerations, flared steel end sections (Figure 10.30) can be furnished for shop fabricated pipe. Such end sections are bolted directly to the pipe. Pre-manufactured end sections are further described in Chapter 2. Precast concrete headwall sections that include a corrugated steel pipe stub can also be specified to protect and enhance pipe ends.

Figure 10.30 Corrugated steel pipe arch flared steel end sections.

When specified, ends of corrugated steel structures can be cut (beveled or skewed) to match the embankment slope as seen in Figure 10.31. However, as indicated in Chapter 7, cutting the ends destroys the ability of the end portion of the structure to resist ring compression forces. Thus, ends with severe cuts must be reinforced, particularly on larger structures. For more complete information see Chapter 7. Cut ends are usually attached to headwalls or ring beams with 3/4” diameter anchor bolts spaced at about 18 inches. (See Chapter 2)

■ **Figure 10.31** Multiple line CSP beveled ends.

The maximum angle permissible for un-reinforced skew cut ends is dependent on the pipe span (or for multiple runs, their combined span) as well as the fill slope. Greater spans or steeper fill slopes limit the degree of skew that can be used without being reinforced with concrete headwalls or ring beams. For larger span structures and multiple structures, this limit is viewed in regard to maintaining a reasonable balance of soil pressures from side-to-side, perpendicular to the structure centerlines.

During backfill and construction of headwalls, the pipe ends may require temporary bracing to prevent excessive distortion. The embankment slope around the pipe ends can be protected against erosion by the use of a headwall, a slope pavement, engineered soft or hard erosion protection, stone riprap, or bags filled with dry sand-cement mixture. Steel sheeting, welded wire, bin-type retaining walls or gabion headwalls may also provide an efficient, economical solution.

Construction Supervision and Control

As in all construction activities, the owner should assign a knowledgeable member of the team to supervise the work in progress, and an inspector to ensure the installation is being performed to specification or accepted practice.

Standard small CSP culverts (6 inches to 60 inches diameter) should be checked at the foundation, bedding, haunches, spring line and minimum cover stages. Generally, con-

SOIL-STEEL BRIDGE STRUCTURE
CONSTRUCTION CONTROL FORM

Owner ______________________ Location ______________________

Supervising Engineer and/or Auth./Rep. ______________________

Contract Firms and Supervising Personnel ______________________

____

Design Engineer ______________________

Geotechnical Assessment ______________________

Stage Inspection	Dates of Inspection	Action-Date and Time of Stage Approval	Authorization to Next Stage
1. Foundation			
2. Bedding			
3. Erection			
4. Backfill-Haunches			
5. Backfill to Spring Line			
6. Backfill to Crown			
7. Backfill to Min. Cover			

Note:

It is suggested that the above form be attached to the certificate of final inspection, and that "as-constructed" drawings be based on cross-section and deflection surveys at least six months after reaching profile grade. (Note: This is a typical control document only.)

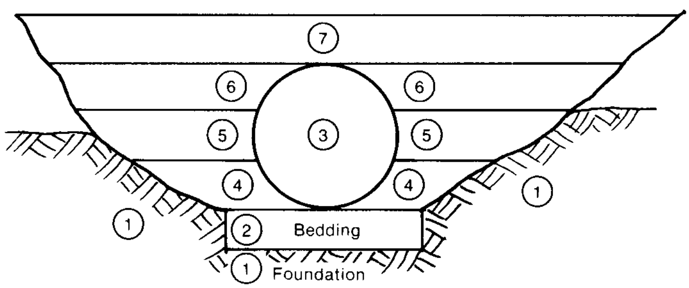


Figure 10.32 Typical inspector's document for construction control of large corrugated steel pipe structures.

struction records need not be kept for CSP in this size range. Larger CSP (72 inch diameter and larger) and Structural Plate Pipe should have inspection at all stages of assembly and installation. Documentation of approval by the authorized inspector should be provided for each stage of construction. Stage inspection means that the contractor is required to have work inspected at specific points of progress, and to secure authorization to proceed to the next stage, in writing. A typical stage inspection form is shown in Figure 10.32.

Soil-steel structures with spans greater than 20 feet should have knowledgeable, on-site inspection personnel, authorized to accept or reject procedures or equipment. These engineered structures should be accorded the same degree of inspection and control as is given conventional bridge construction, which is recognized universally as a specialized discipline in engineering and contracting.

DETENTION, RETENTION AND RECHARGE STRUCTURES

Introduction

Foundation, trenchwall, bedding and backfill considerations for multiple barrel detention systems are not unlike those for conventional CSP installations. However, placement and compaction considerations differ substantially. Construction often must proceed in a different manner making the use of different materials and methods advisable to achieve a sound, economical result. While this design manual covers many of the procedures that must be followed, there may be cases that require additional considerations. It is always good practice to consult with the manufacturer prior to the installation of these systems.

The following are areas that should be considered and planned for each system installed:

- Foundation
- Bedding
- In-situ trench wall
- Backfill material
- Backfill placement
- Construction loading

Foundation Considerations

A stable foundation must be constructed prior to the placement of the bedding material (Figure 10.32). It is important that the foundation is not only capable of supporting the design load applied by the pipe and it's adjacent backfill weight, but is also capable of maintaining its integrity during the construction sequence.

When soft or unsuitable soils are encountered, corrective measures must be taken. The unsuitable material needs to be removed down to a suitable depth and then built up to the appropriate elevation with a suitable structural backfill material.

It is important to make sure that this added structural fill material has a gradation that will not allow the migration of fines, causing possible settlement of the detention system or the pavement above. In cases where the structural fill material is not compatible with the underlying soils, an engineering fabric can be used as a separator.

Figure 10.33 Detention system installation.

The foundation subgrade should be graded to a uniform or slightly sloping grade prior to the placement of the bedding material. If the subgrade is a clay or is relatively non-porous and the construction sequence will last for an extended period of time, it is best to slope the grade to one end of the system. This will enable excess water to be drained quickly, preventing saturation of the subgrade.

Bedding Considerations

A well-graded granular material placed a minimum of 4 to 6 inches in depth works best for the bedding (Figure 10.34). If construction equipment is expected to operate for an extended period of time on the bedding, an engineering fabric can be used to make sure the bedding material maintains its integrity.

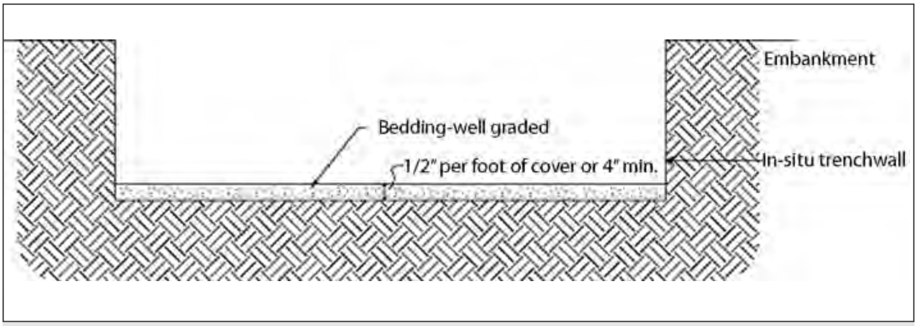


Figure 10.34 Bedding considerations

The use of an open graded bedding material is acceptable; however, an engineering fabric separator is required between the bedding and the subgrade. The bedding should be graded to a smooth consistent uniform grade to allow for the placement of the pipe on the proper line and grade.

In-Situ Trench Wall Considerations

In the event that excavation is required to get the pipe placed on the proper line and grade, consideration needs to be given to the quality of the surrounding in-situ soil (Figure 10.35). The trench wall must be stable and capable of supporting the load that the pipe sheds as the system is loaded. Soils that are weak and not capable of supporting these loads will allow the pipe to deflect excessively. A simple soil pressure check will provide the designer with the applied loads that can be used to determine the limits of excavation required beyond the spring line of the outermost pipes. It should be noted that in most cases, the requirements for providing a safe work environment and enough space for proper backfill placement and compaction, take care of this concern.

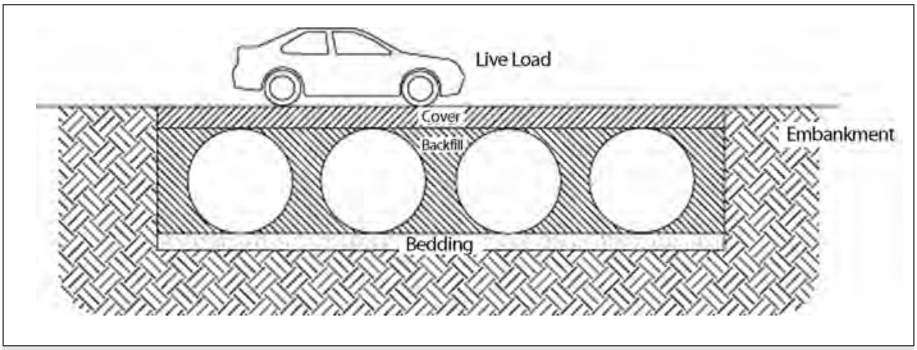


Figure 10.35 In-Situ trench considerations

Backfill Material Considerations

All other considerations aside, the best backfill material is an angular, clean, well-graded granular fill meeting the requirements of AASHTO A-1-a. However, other backfill types can be used (consult the manufacturer). If a uniformly graded (particles all one size) bedding is used, then a geotextile separation fabric should be used to prevent the migration of fines (Figure 10.36).

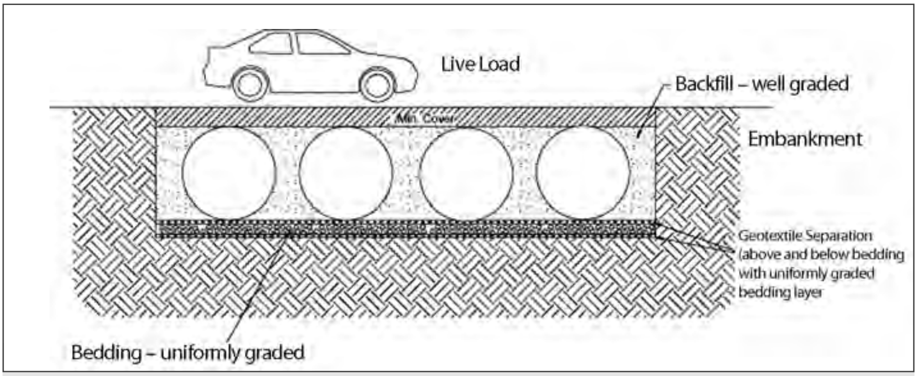


Figure 10.36 Backfill considerations

Depending on the size of the pipe and the spacing, it is at times desirable to use a uniformly graded material for the first 18 to 24 inches. This type of material is easier to place under the haunches of the pipe and requires little compaction effort. In the event that this type of material is used, then a separation geotextile should be used above and below these initial lifts, depending again on the bedding material (Figure 10.37).

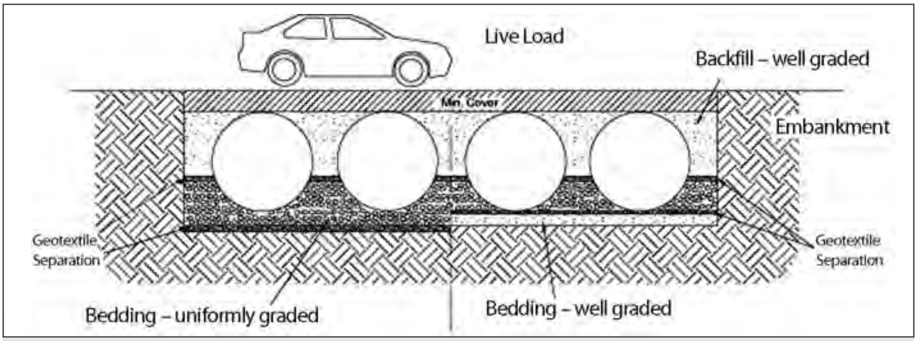
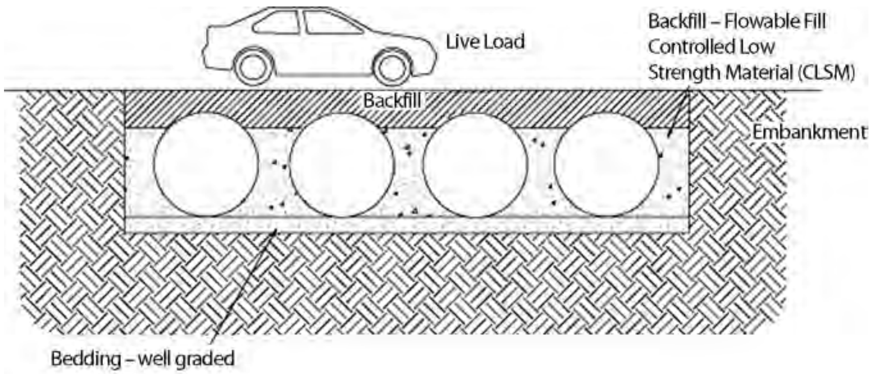


Figure 10.37 Geotextile separator recommendations

It is not desirable to use an open graded fill beyond the initial 18 to 24 inches because the proposed fill often does not provide adequate confining restraint to the pipes in these types of systems.

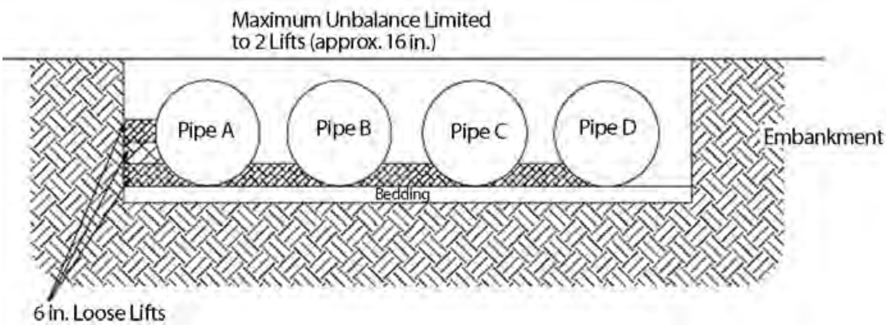


■ **Figure 10.38** Backfill with CLSM

Figure 10.38 shows backfill with CLSM, another suitable material.

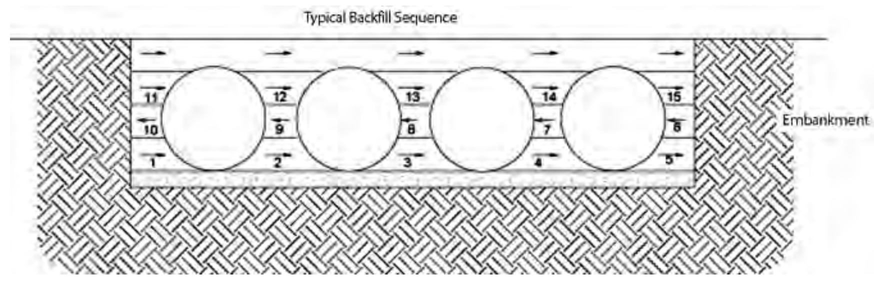
Backfill Placement Considerations

The backfill should be placed in 6 inch loose lifts and compacted to 90% AASHTO T99 standard proctor density (Figure 10.39). The backfill must be placed in a balanced manner making sure that no more than a two-lift differential is present from one pipe side to the other during the backfilling process. Excessive backfill differential heights from one side of the pipe to the other can cause pipe distortion or lateral movement.



■ **Figure 10.39** Backfill placement

As backfill is placed between the pipes it must be kept balanced from side to side as well as advanced at the same rate along the length of the detention system. In other words, if you place the first lift between pipe A and B for a distance of 25 feet along the length of the system, then 25 feet of fill needs to be placed between pipes B and C and so forth until all pipes are backfilled equally (Figure 10.40).



■ **Figure 10.40** Backfill placement sequence

For large systems, conveyor systems have been used to place the fill effectively. Backhoes with long reaches or draglines with stone buckets have also been used effectively to place the fill along the pipe lengths until minimum cover is reached for construction loading across the entire width of the system. On long parallel sections of pipe, the contractor may need to backfill in stages along the pipe lengths. Once the required cover is reached on the initial section, then the equipment advances forward to the end of the recently placed fill and the sequence begins over again until the system is completely backfilled. This type of construction sequence will provide room for stockpiled backfill directly behind the backhoe as well as for the movement of construction traffic. Material stockpiles on top of the backfilled detention system should be limited to 8-10 feet maximum height and must provide balanced loading across all barrels. To determine the proper cover over the pipes to allow the movement of construction equipment, see the section that follows, **Construction Loading Considerations**.

The trench width and pipe spacing requirements were established to allow the full range of backfill materials to be used. These spacings can be reduced when special backfill and special care is used. The limit is where the difficulty of access for assembly and backfill compaction becomes uneconomical.

Reducing the spacing between pipes can be especially helpful where the multiple runs often involved with detention, retention and recharge systems are encountered. These are typically low cover applications where the strength of the backfill is less important and high compaction not as critical. Clean, non-plastic, easily flowing backfill materials have higher strengths than other backfill materials, even at lower compaction levels.

Spacings of 24 inches are generally not objectionable. A spacing of 18 inches or less can be used with backfill materials such as crushed rock, # 57 stone or pea gravel. These materials are more easily placed into the haunch. When necessary, concrete vibrators can be used to move and consolidate the backfill much like they do fluid concrete, to assure there are no voids left. Alternatively conventional vibratory compaction plates have been used inside the pipe invert to help move and consolidate these materials against the outside of the pipe.

Low strength grout, controlled low strength materials (CLSM), etc. allow spacing of as little six inches if the pipes can be joined. However, flotation becomes a special consideration and may require the pipe to be weighted (Figure 10.41).

Flotation

When CSP reline pipes or those backfilled with grout are installed, a primary consideration is the need to control flotation. Fluid grout, which may have a density of 120 pcf or greater can develop greater buoyancy forces than water. To minimize flotation problems, grout is typically placed in thin lifts from side to side of the pipes in a balanced manner.

Direct burial pipes typically are more difficult to hold down. Methods that have been used to provide a degree of hold down restraint include placing timbers over the pipe with each end wedged into the trench wall, or placing tension straps over the pipe crown tied to earth anchors in the foundation. Where feasible, pipe have been filled with water or weighted down with concrete blocks placed on roller dollies in the invert.

Where the hold down restraints are intermittent, support spacing limits apply such as discussed for aerial spans in Chapter 8. However, it must be recognized that the aerial span limits apply to water filled pipes whereas inundating the entire pipe with grout could develop roughly twice the uplift, due to the higher grout density.

One way to reduce the buoyant forces is the use of lightweight cementitious backfill materials. These are often simply portland cement, water and a foaming agent that, at 30 to 40 pounds per cubic foot, provide excellent backfill and lower buoyancy forces than low strength grout. While these special backfill are more costly, the closer pipe spacings reduce the necessary quantity.

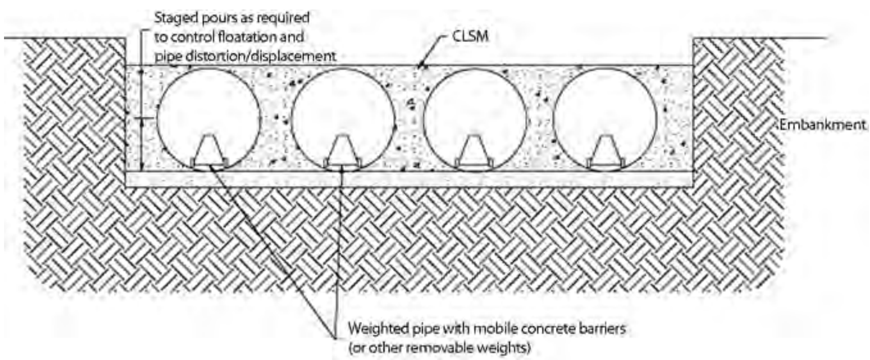
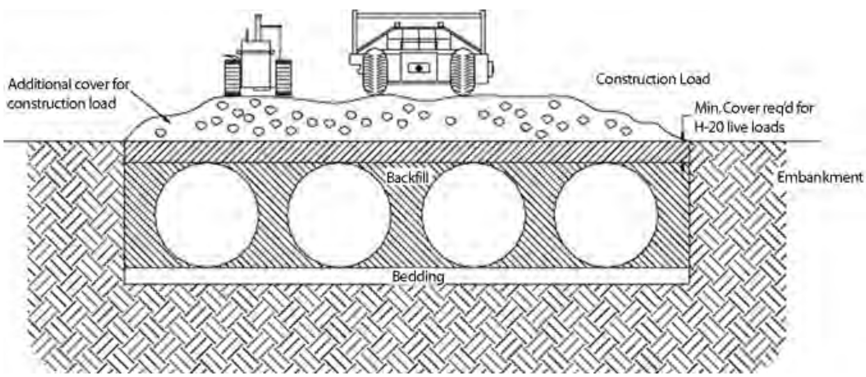


Figure 10.41 Stage pours for CLSM placement

■ Box culvert with construction equipment loading on top of the structure.

Construction Loading Considerations

Typically, the minimum cover specified for the project is for standard AASHTO H-20 live loads. Construction loads can greatly exceed those loads for which the pipe is designed in its completed state. In many cases, increased temporary minimum cover requirements are necessary to facilitate construction loading (Figures 10.26 and 10.42). Since construction equipment varies from job to job, it is best to discuss the minimum cover requirements during construction with the contractor at the preconstruction meeting. Table 10.1 provides guidelines.



■ **Figure 10.42** Additional cover for construction loading

Special Considerations

Since most of these systems (detention, retention, and recharge structures) are constructed at a grade below elevation for the surrounding site, rainfall can cause the excavation to fill with water rapidly. This rapid influx of water can potentially cause floatation and movement of the previously placed pipes. To help mitigate potential problems, it is best to start the system at the outlet or down stream end with the outlet already constructed to allow a route for the water to escape. Temporary diversion measures to handle flow may be required due to the restricted nature of the outlet pipe.

FIELD ASSEMBLED STRUCTURAL PLATE STRUCTURES

Structural Plate Corrugated Steel Pipe (SPCSP) differs from shop fabricated pipe in that the structure is shipped in unassembled steel plates to the jobsite. Structures larger than what can traditionally be shipped are easily assembled at the project site. Structural plate structures have an advantage over shop fabricated pipe in that the steel plates that comprise them can be made from thicker material and with deeper corrugation profiles. Standard SPCSP structures are those that are comprised of a 6 inch x 2 inch, 15 inch x 5 1/2 inch or 16 inch x 6 inch corrugation profiles and do not fall under the long span category.

Unloading and Handling

Plates for structural plate structures are shipped nested in bundles complete with bolts and nuts and the assembly drawings and instructions necessary for erection. Bolts are color coded for length identification. Bolts for every SPCSP structure are provided in two lengths. The longer length is required when three or four thicknesses of plate overlap.

Bundles are sized so that cranes, loaders, or other construction equipment already on the job are all that is needed for unloading. Normal care in handling is required to keep the plates clean and free from damage by rough treatment. Pre-sorting the plates as they are unloaded, on the basis of their radius and location in the structure is important. All plates are clearly marked so they can be easily sorted.

Assembly Methods

A variety of assembly techniques are available to suit site conditions, as well as the size or shape of the structure. Maintaining the design shape must be a key objective during plate assembly.

There are four basic methods by which structural plate structures can be assembled:

1) Plate-by-Plate Assembly - The majority of SPCSP structures are assembled directly on the prepared bedding or footings in a single plate-by-plate erection sequence, commencing with the invert, then the sides, and finally, the top. This method is suitable for any size of SPCSP structure. Initially, structures should be assembled with as few bolts as possible. The curved surface of the nut is always placed against the plate. Three or four finger-tightened bolts near the center of each plate, along longitudinal and circumferential seams, are sufficient to hold the assembly in place. This procedure gives maximum flexibility until all plates are fitted into place.

   After part of the structure has been assembled into its shape by partial bolting, the remaining bolts can be inserted and hand tightened. Always work from the center of a seam toward the plate corner. Alignment of bolt holes is easiest when bolts are loose.

   After all the bolts are in place, tighten the nuts progressively and uniformly, starting a few rings behind the stair-stepped plate assembly. The operation should be repeated to be sure all bolts are tight.

   If the plates are well aligned, the torque applied with a power wrench need not be excessive. A good fit of the plates is preferable to the use of high torque. Bolts should not be over tightened. They should be torqued initially to a minimum of 150 foot pounds and a maximum of 300 foot pounds. It is important that the initial torquing be done properly. In many structures, nuts may be on the outside, and re-torquing would not be possible after backfill.

   In some applications, such as for pedestrian and animal underpasses, it is specified that all bolt heads should be on the inside of the structure, for safety and visual uniformity. If a paved or gravel invert is to be placed, it may be desirable to have the bolt ends protruding into the area to be covered.

   After backfilling, the structure relaxes and the actual in-service bolt torque will decrease slightly. Depending on plate and structure movements, some bolts may tighten, and some may loosen over time. The degree of change in torque values is a function of metal thickness, plate match, and change of structure shape during backfilling. This is normal and not a cause for concern, should checks be made at a later stage.

2) Component Sub-Assembly - This is the pre-assembly of components of a ring, away from the bedding (Figure 10.43). The components are usually comprised of preassembled sections of the bottom plates, the side plates and the crown plates. This method is suitable for most soil-steel bridge installations. Component sub-assembly is often more efficient than the plate-by-plate

method. Its main advantage is that it permits simultaneous progress at more than one location at the site. The final assembly operation can be carried out at the same time as the sub-assembly operation.

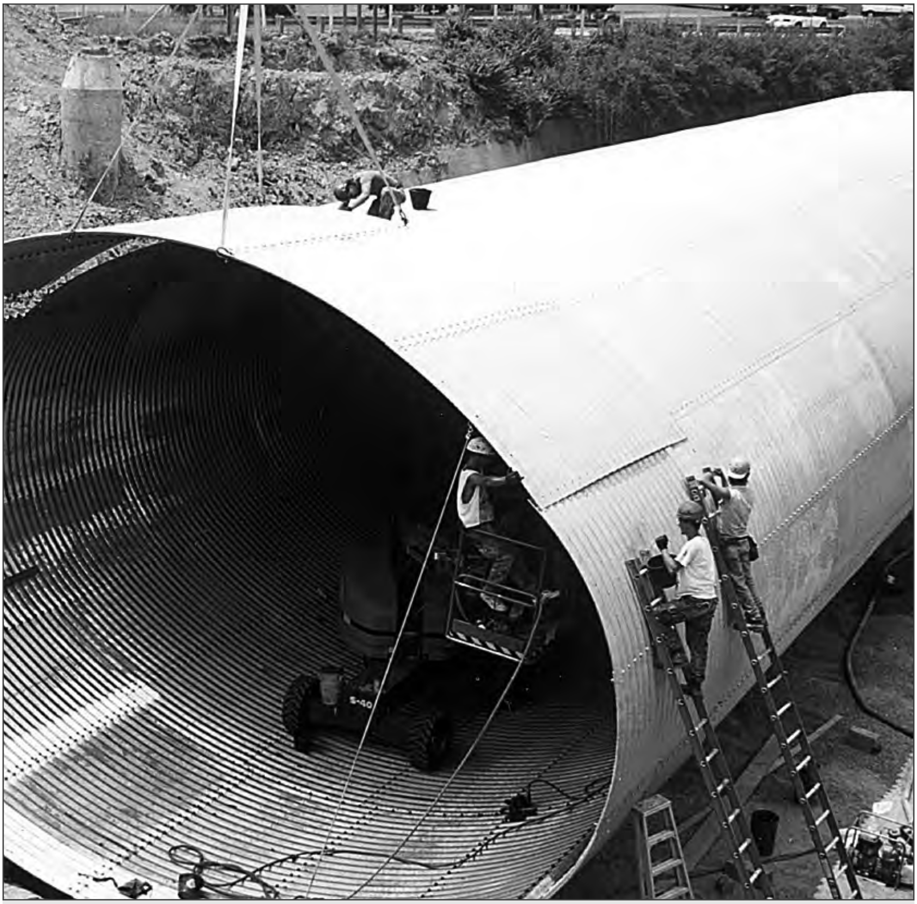
Placing the invert components on a prepared shaped bedding poses a problem with bolt insertion and torquing for large radius inverts (i.e. pipe arch or horizontal ellipse in particular). Bolts can be pre-placed by the use of spring clips. Other methods, such as the use of magnets or access trenches, may be used. Experienced assemblers often preassemble sections of invert plates prior to placement, as long as this does not affect the placement of side and top plates.

During component assembly of larger SPCSP structures, it is important to maintain curvature and resist flattening due to torquing and weight of the sections. The invert component should be sized to the proper radius and chord length before the side assemblies are started. This can be controlled by horizontal sizing cables. As the side components are bolted in place, these cables should be moved to the spring line. Similarly, the sides should be held to the design shape, to effect top closure. When design shape is maintained during erection, the top sub-assembly should fit into place.

The sizing cables should be left in place until all other bolts are torqued. It is important that design shape and size be maintained throughout the backfill operation, with allowances for normal movement arising from backfill pressures.

The bolts in plate assembly components should be fully tightened prior to placement. This means that loose-bolting until the full ring is completed, is not possible. Therefore, it becomes important that components being bolted are aligned before torquing. Shape checks should be carried out during and after erection to be certain that the erected shape is within design tolerances. Necessary shape corrections must be carried out prior to proceeding with the backfill operation.

Additional bolt tightening may be required on large structures. Corner bolts control position, and the remaining nuts are torqued to mid-range (approx. 120 foot pounds). Once the structure is completed, and correct alignment of plates is assured, another pass may be made to fully torque to not more than 300 foot pounds before the next ring assembly is completed.



■ **Figure 10.43** Structure being completed with preassembled components.

3) Pre-Assembly of Sections - In this method, circumferential rings of round structures are assembled off-site. These rings, or cans, are then transported to the assembly site for connection along their circumferential seams. The end corrugations of one ring must be lapped with those of its adjoining ring, to provide continuity in the assembly.

4) Complete Pre-Assembly - Pre-assembly of the complete structure can be done either at the factory or at the jobsite. The factory pre-assembled method is used for relatively small span installations, this application being limited by shipping size. The field pre-assembly method is selected for structures to be lifted intact or to be skidded onto a prepared foundation and bedding. Pre-assembly techniques are essential for installation under submerged bedding conditions (Figure 10.44).

■ **Figure 10.44** Assembled structure being placed in position.

Special Assembly Techniques

Structural Plate Arches

Structural plate arch shapes differ from other plate structures in that the edges of the arch are erected on an abutment, or footing. The arch footings are usually constructed of poured-in-place concrete, but may also be timber sills or steel footing pads. The use of piling is not recommended, as this will introduce an unyielding foundation. If the entire soil-steel arch structure is allowed to settle with the foundation, this will avoid drag down loads on the arch and encourage positive soil arching and interaction.

The unbalanced steel channel on which the bottom plates rest must be located accurately as per the design drawings to ensure proper and easy plate assembly. Care must be taken to insure that the pre-punched holes in the two opposing channels are in accurate alignment. The installer must remember to cast the unbalanced channel at the correct angle and slope to accommodate the bottom plates. Improper placement of base channels can create serious problems in arch construction.

The layout for channel installation should be shown on the fabricator's plate assembly drawings. If accurate structure overall length is important, as it may be in pre-locating concrete headwalls, the designer should remember that the actual overall length is the net length plus 4 inches, due to the lips at the end of the end plates. Pre-locating headwalls is not a recommended practice due to the need to shape and support the headwall opening. This is further complicated by the flexible nature of these structures combined with manufacturing tolerances.

Scaffolding or temporary bracing of the early rings is usually necessary with the arch shape, as the initial plates are not self-supporting. Component pre-assembly is often advantageous.

Structural Plate Pipe Arches

During the assembly of multiple radius structures such as pipe arches, underpasses and ellipses, care must be taken to ensure proper assembly and plate laps. Where different radius plates meet at a longitudinal seam, it may take extra effort to fully seat the corrugations and obtain the tangent plate lap required. Properly shaped bedding is especially important to assembly.

Pipe arches are currently fabricated in two forms. Some have multiple radius corner plates that include both corner and top radius elements. Others use separate corner and top plates with a longitudinal seam at this juncture. The plate lap arrangement differs with this type of fabrication. The manufacturer's assembly instructions should be followed to avoid improper plate laps.

Other Structural Plate Considerations

Asphalt Coating - Shop or Field Applied

Where structural plates require a protective coating in addition to galvanizing, there are suitable materials available for application to the components, to the assembled structure in the field, or on pre-assembled structures in the plant. Plates must be clean and dry. The coating can be asphalt mastic containing mineral fillers and stabilizers sprayed on under high pressure to a minimum thickness of 0.05 inches (AASHTO M-243 / ASTM A849).

Seam Sealants

A degree of leak resistance for SPCSP structures can be achieved with modern seam sealants. Standard SPCSP structures, because of the bolted construction and lapped plates, are not intended to be watertight. On occasion, where a degree of water-tightness or prevention of soil infiltration is required, it is practical to insert a seam sealant tape within the bolted seams. The seam sealant normally specified is wide enough to cover all rows of holes in plate laps, and of the proper thickness and consistency to effectively fill the voids in plate laps.

The procedure for installing seam sealant is as follows:

1) The tape is rolled over each of the surfaces that will come in contact and worked into the corrugations. The tape should not be stretched.
2) Any paper backing must be removed prior to placing the lapping plates.
3) At all points where three plates intersect, an additional thickness of tape is placed for a short distance to fill the void caused by the transverse seam overlap.
4) A hot spud or a sharp tool dipped in machine oil is used to punch through the tape to provide a hole for inserting the bolts.
5) Tightening of the bolts twice is usually necessary to maintain adequate torque. As the seam sealant creeps under the pressure, final bolt torque will be lost. This is expected and not a concern. Plate fit-up and proper meshing is most important.

Backfill and Compaction for Standard Structural Plate Sizes and Shapes

All of the backfill and compaction principles for CSP apply for structural plate with some additional considerations. Because the large structures are more flexible, shape control is especially important. The manufacturer of the structural plate product should always be contacted for additional information regarding backfill and compaction of structural plate structures.

Backfill Material for Steel Structural Plate

Granular-type soils should be used as structural backfill (the soil envelope next to the metal structure). The order of preference of acceptable structure backfill materials is as follows:

1) Well-graded sand and gravel; sharp, rough, or angular if possible.
2) Uniform sand or gravel.
3) Mixed soils (not recommended for large structures).
4) Approved stabilized soil.

The structure backfill material should conform to one of the soil classifications from AASHTO Specification M-145 meeting the requirements of A1, A2 or A3. For heights of cover less than 12 feet, A-1, A-3, A-2-4 and A-2-5 or approved stabilized soils are recommended for long spans and box culverts only A1, A2-4, A2-5 and A3 are allowed. For heights of cover of 12 feet or more, A-1 and A-3 are suggested. For all structures with covers exceeding 20 feet requirements of A1 or A3 are desired.

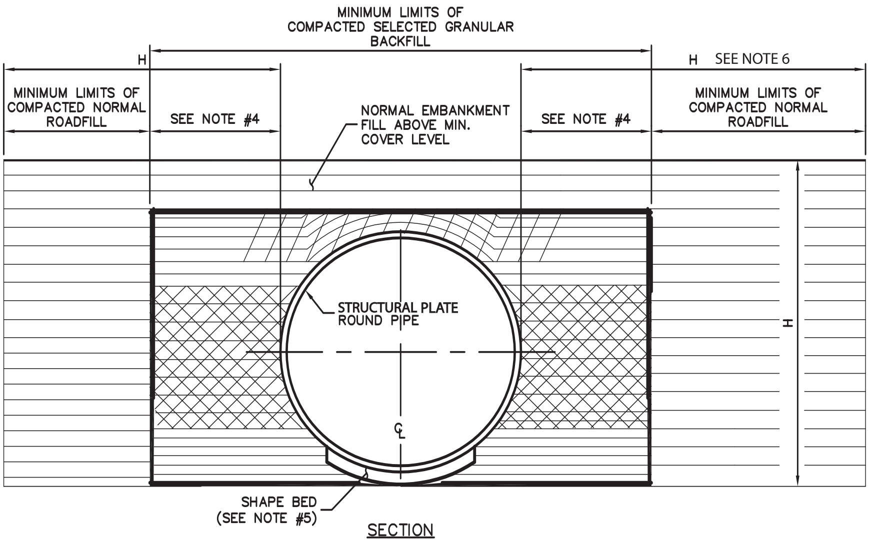
The extent of the structure backfill zone is a function of the pressures involved and the quality of the foundation soils, the trench wall or embankment soil, and the fill over the structure. Figure 10.45 shows a typical backfill envelope.

Arch Structure Backfill

Care must be taken in backfilling arches, especially taller arches (when the rise is greater than the span), because they have a tendency to shift sideways or to peak under backfilling loads. The ideal way is to cover an arch in layers with each layer conforming to the shape of the arch. If one side is backfilled more than the other, the arch will move away from the larger load. If both sides are backfilled equally and tamped thoroughly, the top of the arch may peak unless enough fill has been placed over it to resist the upward thrust. These precautions apply also to other corrugated steel structures, but to a lesser degree.

If the headwalls are built before the arch is backfilled as recommended, the backfill material should first be placed adjacent to each headwall, placing and compacting material uniformly on both sides of the structure until the top of the arch is reached. Then backfill should proceed toward the center by extending the ramp with care being taken to place and compact the material evenly on both sides of the arch. Top loading a small amount of backfill material will help prevent peaking.

When backfilling arches without headwalls or before headwalls are placed, the first material should be placed midway between the ends of the arch forming as narrow a ramp as possible until the top of the arch is reached. The ramp should be built evenly from both sides and the backfill material should be thoroughly compacted as it is placed. After the two ramps have been built to the depth specified to the top of the arch, the remainder of



CRITICAL BACKFILL ZONE, PRESSURE ON SOIL GREATEST HERE.

INITIAL LIFTS OVER CROWN OF STRUCTURE AS INDICATED BY SHADED AREA TO BE COMPACTED TO REQUIRED DENSITY WITH HAND OPERATED EQUIPMENT OR WITH SMALL TRACTOR (D–4 OR SMALLER) DRAWN EQUIPMENT.

SELECT GRANULAR STRUCTURAL BACKFILL LIMITS.

NOTES:

1. ALL SELECT GRANULAR BACKFILL TO BE PLACED IN A BALANCED FASHION IN THIN LIFTS (6 in. - 12 in. LOOSE TYPICALLY) AND COMPACTED TO 90 PERCENT DENSITY PER AASHTO T–99.
2. COMPLETE AND REGULAR MONITORING OF THE STUCTURAL PLATE SHAPE IS NECESSARY DURING ALL BACKFILLING OF THE STRUCTURE.
3. PREVENT EXCESSIVE DISTORTION OF SHAPE AS NECESSARY BY VARYING COMPACTION METHODS AND EQUIPMENT.
4. GREATER OR LESSER DISTANCE MAY BE REQUIRED. DISTANCE DEPENDS ON BEARING LOAD FOR ANY GIVEN LOADING, STRUCTURE SHAPE AND BACKFILL MATERIAL. THIS MUST BE EVALUATED BY THE PROJECT ENGINEER FOR EACH SPECIFIC SITUATION.
5. SHAPED BED FOR A MINIMUM WIDTH OF SPAN/2. MINIMUM BEDDING THICKNESS IS TWICE THE CORRUGATION DEPTH.
6. EMBANKMENT WIDTH H TO BE SUCH THAT A STABLE EMBANKMENT CAPABLE OF RESISTING SIDE PRESSURES FROM STUCTURAL PLATE SHAPE WILL BE MAINTAINED THROUGHOUT THE LIFE OF INSTALLATION. THIS WIDTH TO BE DETERMINED BY THE PROJECT ENGINEER.

Figure 10.45 Typical structural plate backfill envelope the backfill should be placed and compacted by extending the ramp both ways from the center to the ends and as evenly as practical on both sides of the arch.

SPECIAL STRUCTURAL PLATE SHAPES AND CORRUGATIONS

Special structural plate shapes and corrugations are those that don't fall under the standard plate category. Generally the manufacture and design of these structures are proprietary and the individual manufacturers should be contacted for further information regarding their products. This section is intended to be an overview of the special considerations typically needed in regards to these structures.

Long Span Structures

Long span structures are unique from standard structural plate in two ways: longer spans and/or deeper corrugations. Long spans are available in spans up to 75 feet. Plate erection may differ from the recommendations for standard structures with added attention given to maintaining structural shape during assembly and backfill. Proper backfill materials and compaction are essential to structural integrity and should comply with instructions given under backfilling. Additional construction supervision and control is required for long span structures.

Foundation

Long span structures are relatively light in weight and often have significant rise dimensions. Unless cover is significant, they exert lower bearing pressures on the foundation than the structural backfill materials beside the structure. Foundation bearing strength requirements generally relate to the need to support the side-fill without excessive settlement. If any relative settlement occurs, it is preferable that the structure settles relative to the side fill to avoid developing increased loads as a result of negative soil arching.

When a structure with a bottom is used, plates have relatively larger radii and exert limited pressure on the foundation. It is often only necessary to provide a uniform, stable foundation beneath the structure to support erection activities. For arch structures, footing designs must recognize the desired relative settlement conditions. The need for excessively large footings or pile supports is indicative of poor soil conditions and therefore, inadequate support beneath the side-fill.

Bedding

Pipe arch, horizontal ellipse and underpass shapes with spans exceeding 12 feet should be placed on a shaped bed. The shaped area should be centered beneath the pipe and should have a minimum width of one-half the span for pipe arch and underpass shapes, and one-third the span for horizontal elliptic shapes. Preshaping may consist of a simple "V" graded into the soil.

Backfill

While basic backfill requirements for long span structural plate structures are similar to those for smaller structures, their size is such that excellent control of soil placement and compaction must be maintained to fully mobilize soil-structure interaction. A large portion of their full strength is not realized until backfill (side-fill and overfill) is in place.

Of particular importance is control of structure shape. Equipment and construction procedures used should ensure that excessive structure distortion will not occur. Structure shape should be checked regularly during backfilling to verify acceptability of the construction methods used. The manufacturer will specify the magnitude of allowable shape changes.

The manufacturer should provide a qualified construction inspector to aid the engineer during all structure backfilling. The inspector should advise the engineer on the acceptability of all backfill materials and methods and undertake monitoring of the shape.

Structural backfill material should be placed in horizontal uniform layers not exceeding 8 inches thick before compaction and should be placed uniformly on both sides of the structure. Each layer should be compacted to a density not less than 95% per (Standard Proctor Density). The structure backfill should be constructed to the minimum lines and grades shown on the plans. Permissible exceptions to the structural backfill density requirement are: 1) the area under the invert; 2) the 12 to 18 inches width of soil immediately adjacent to the large radius side plates of high profile arches and inverted pear shapes; 3) and the first horizontal lift of overfill carried ahead of and under construction equipment initially crossing the structure.

Box Culverts

Box culverts are treated differently than soil steel structures. They are very stiff compared to long span structures and this makes the placement and compaction of backfill materials easier.

Assembly of Box Culverts

Due to the stiffness requirements of a box culvert shape, some installations may require the addition of reinforcing ribs. The box culvert manufacturer should be consulted prior to assembly to insure the proper technique is followed for installation.

Backfill of Box Culverts

Box culverts require long span backfill materials (above) that are properly compacted in a zone that extends 3 feet on each side of the outside of the box and up to the minimum cover. The granular backfill material in the engineered backfill zone should be placed uniformly on both sides of the box culvert in layers not exceeding 8 inches in depth and compacted to a minimum of 95% Standard Proctor Density (ASTM D698). Compaction testing during construction is the responsibility of the contractor. The difference in the levels of backfill on the two sides, at any transverse section should not exceed 2 feet. The range of cover over steel box culverts is from 1.4 to 5 feet.

Heavy compaction equipment or backfill dump trucks that could alter the shape of the box culvert should be avoided. Heavy compaction equipment should not be allowed within 3 feet of the structure wall or close enough to cause distortion.

A non-woven geotextile should be placed at the ends of hollow or corrugated reinforcing ribs to prevent backfill from entering the cavity between the barrel and the reinforcing rib.

CHAPTER SUMMARY

Proper installation of any drainage structure will result in longer and more efficient service. This installation and construction chapter is intended to call attention to both good practice and to warn against possible pitfalls. The principles apply to most drainage pipe materials. It is not a specification but an aid to your own experience.

The following items should be checked to insure proper installation:

(1) Check alignment and grade in relation to stream bed.
(2) Make sure the length of the structure is correct.
(3) Excavate to correct width, line and grade.
(4) Provide a uniform, stable foundation.
(5) Unload and handle structures carefully.
(6) Assemble the structure properly.
(7) Use a suitable backfill material.
(8) Place and compact backfill as recommended.
(9) Protect structures from heavy, concentrated loads during construction.
(10) Proper end treatment placement can protect the soil at the ends of culvert from erosion.
(11) Construction supervision should be considered for all installations, but most especially for the more critical or complex applications.
(12) Review additional considerations for large or deeply buried structural plate structures

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■ **Figure 11.1** 48 inch diameter CSP being used to slip line failed 54 inch diameter concrete pipe. Class 5, on partially constructed PA DOT Project – Southern Expressway – near the Pittsburgh Airport.

CHAPTER eleven

Value Engineering and Life Cycle Cost (LCC)

INTRODUCTION

This chapter deals with the important subject of cost efficiency. This requires a working understanding of both value engineering and life cycle cost. Value Engineering (VE) is the critical first step to insure that correct alternates are considered in the life cycle cost (LCC). Otherwise, the engineer may be comparing apples and oranges.

This chapter offers guidelines for designing corrugated steel pipe systems that are structurally adequate, hydraulically efficient, durable and easily maintained. By following these guidelines, equal or superior performance can be realized through use of CSP products. The basic techniques of Value Engineering apply. By allowing design and bid alternates, including the proper corrugated steel pipe system, savings on the order of 20% can frequently be realized. Alternative designs offer even more promise, with savings of as much as 90% possible compared to the costs of conventional design. Thus, innovative use of corrugated steel pipe design techniques can offer truly substantial savings, with no sacrifice in either quality or performance.

VALUE ENGINEERING

Value Engineering is defined by the Society of American Value Engineering as: "The systematic application of recognized techniques which identify the function of a product or service, establish a value for that function and provide the necessary function reliably at the lowest overall cost." In all instances, the required function should be achieved at the lowest possible life cycle cost consistent with requirements for performance, maintainability, safety and aesthetics.

Value Engineering is functionally oriented and consists of the systematic application of recognized techniques embodied in the job plan. It entails:

1) Identification of the function
2) Placing a price tag on that function, and
3) Developing alternative means to accomplish the function without any sacrifice of necessary quality.

By contrast, lack of information, wrong beliefs, habitual thinking, risk of personal loss, reluctance to seek advice, negative attitudes, over specifying and poor human relations represent barriers to cost-effectiveness.

Many VE recommendations or decisions are borne of necessity. Often, the limited availability of financial resources, equipment or material, or physical limitations of time and topography, limit the options available. These are the very reasons that Value Engineering came into being. It is a systematic process of obtaining the best result within the available resources. If the appropriate job plan is carefully followed, the alternative selected should be equal if not better, and capable of functioning within the stated limitations.

To be competitive, designers have to be production oriented and quickly prepare completed plans that are practical and economical. A simple technique to achieve efficiency and pursue maximum economy is for the project specifications to include a range of alternative materials, thereby engaging contractor creativity and experience. Of course, designers should always be open to Value Engineering change proposals.

The utility of Value Engineering as a cost control technique has long been recognized by the Federal Government. It was first used by the Navy in 1954. Subsequently, through the action of Congress and the Office of Management and Budget (OMB), virtually any federal agency with an annual budget in excess of $10 million was required to utilize VE analysis. AASHTO has the following position on Value Engineering:

To improve design excellence and achieve cost reduction and quality control, it is AASHTO's position that:

Each member state should establish an ongoing VE program.

- *The challenges of rising costs and diminished resources be addressed through the application of VE principles and practices in project development, construction, traffic operation, maintenance and other appropriate areas.*
- *Guidelines be provided to member organizations to promote and assist in broad acceptance and use of value engineering with the provision of flexibility to adapt to individual needs.*

Value Engineering has become a common practice in most transportation or highway departments in the US and among the federal agencies. It is recognized as an effective approach to obtain best results from limited taxpayer resources.

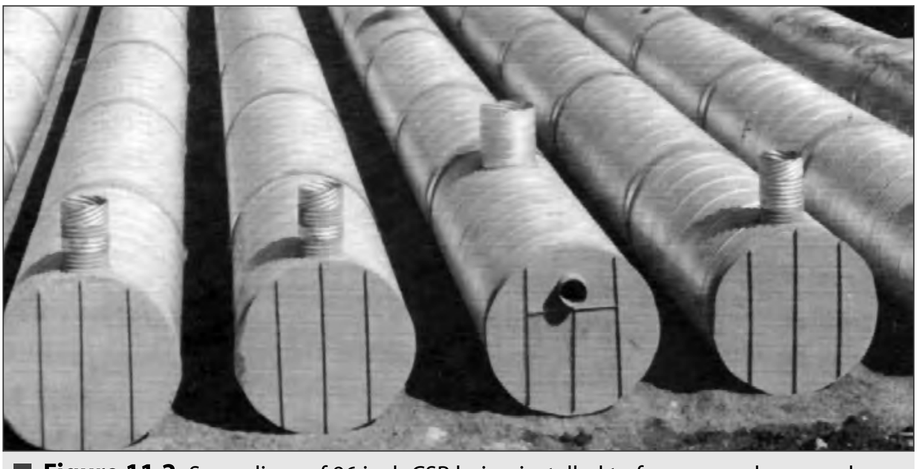


Figure 11.2 Seven lines of 96 inch CSP being installed to form an underground stormwater detention facility.

Inclusions of Alternative Materials in a Project Induce Lower Prices

The fundamental of a free market system is competition. By specifying as many alternative materials as possible, the owner of a project is assured of the most economic project possible since competition encourages lower pricing.

Value Engineering helps allow for competition of alternative materials because it provides a formalized approach that encourages creativity both during the design process and after the bid letting. During the design process it involves the consideration of both alternative products with equal performance and alternative designs. After bid award, it involves the substitution of different project plans together with revised design or materials to meet time constraints, material shortages, or other unforeseen occurrences that would affect either the completion date or quality of the finished product.

Thus, there are two basic ways to use Value Engineering: (1) At the design stage to determine the most cost-effective material or design to specify without alternates; (2) To select the most cost-effective bid submitted on alternates.

In the first case it is important to use Value Engineering principles when calculating estimates for various materials being considered. This means including in the estimates all the factors bidders would consider in their bids. Installation cost differences between concrete and corrugated steel pipe result from pipe dimensions, foundation and bedding, required equipment and speed of assembly. Also, factors affecting public safety and convenience such as detours and total time on job should be considered. In the second case, where alternate bids are taken, it is important to clearly spell out in the plans and specifications the differences in pipe and trench dimensions for concrete and corrugated steel pipe. Foundation, bedding and minimum cover differences may also be significant. Construction time schedule differences could be a factor and should be required to be shown.

Cost Savings in Alternative Designs

In addition to the savings resulting in allowing pipe alternatives in conventional designs, alternative designs based on entirely different water management procedures can offer even more significant savings. One example is in the design of storm water systems that meet environmental requirements in force today. By using these techniques on a total system basis, it is possible to minimize the use of expensive surface lands for ponds, which can be hazardous during flood conditions, and instead store the flood waters underground in large corrugated steel pipe detention chambers as shown in Figure 11.2.

Another excellent example of the application of value engineering principles in a real situation is the use of large diameter CSP as an alternative to bridge replacement. When faced with limited funds and the need to replace two deteriorating concrete flat slab bridges, the Abilene District of the Texas DOT developed an innovative approach.

Utilizing 96 inch diameter pipe at one location and 112 inch x 75 inch pipe arch at the second, special head walls and wing walls and flowable fill to grout all voids, a 51% cost savings was realized:

Remove and Replace Alternative

Class A Concrete	$277,200
Detours, Traffic control	74,000
Remove old structure	30,000
Total Estimated Cost	$381,200

Rehabilitate with CSP

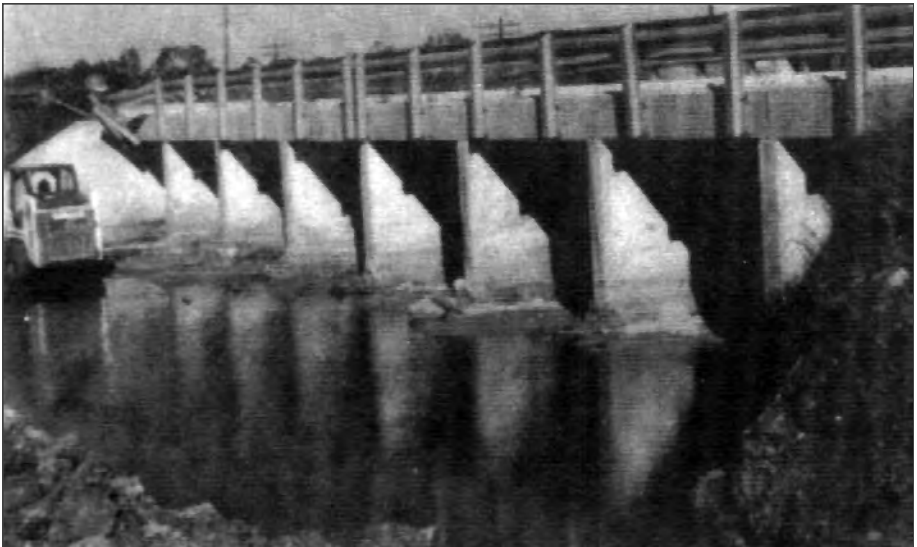
Class A Concrete	$99,550
Corrugated Steel Pipe	78,200
Flowable Fill	7,174
Riprap	2,278
Total Actual Cost	$187,202

Cost Savings

Amount	$193,998
Percent	51%

In addition to the lower cost, the CSP alternative did not impede traffic flow and thus public safety was not compromised.

No detours were necessary, the roadway was widened, and the load carrying capacity was increased. The following photos show how CSP solved the problem.



■ **Figure 11.3** Texas DOT bridge replacement project. Before...

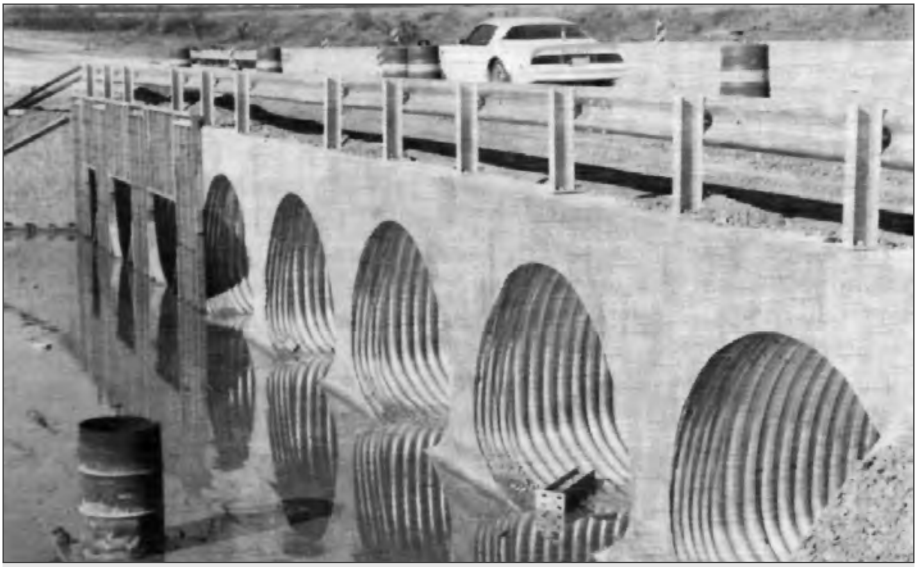


Figure 11.4 Texas DOT bridge replacement project. After...

CSP Products for VE Application

The following list indicates the possible VE applications where the various CSP products can provide a cost-effective solution.

Product	Possible VE Application
All CSP products	Storm Sewers Underground Detention Systems Bridges
Spiral Rib CSP	Hydraulic Storm Sewers Rehabilitation / Reline
Double Wall CSP	Hydraulic Storm Sewers Rehabilitation / Reline
Concrete Lined CSP	Hydraulic Storm Sewers Rehabilitation / Reline
Slotted Drain CSP	Sheet Flow Capture vs. Inlets
Structural Plate	Bridges Stream Enclosures Golf Cart Crossings / Underpasses Underground Detention Systems Special Foundations / Piling

LIFE CYCLE COST

Life Cycle Cost is a technique that compares differing series of expenditures by restating them in terms of the present worth of the expenditures. In this way, competing designs that have differing cost expenditures at different intervals can be compared and the lower cost design chosen on a present worth basis.

The technique is familiar to most engineers and engineering students. Anticipated future costs are discounted by using a present worth factor and restated in terms of today's costs. Once discounted, all the costs for one project design can be added together and fairly compared to all of the costs for a competing project design.

Life Cycle Cost is well suited for comparing the competing bids for culvert and storm sewer projects when pipe material alternatives such as corrugated steel (CSP) and reinforced concrete (RCP) are specified.

The life cycle cost equations are fairly straightforward. Tables can be used to determine the various present worth factors of competing projects or numerous computer programs and hand held calculators are available to solve these problems.

The real difficulty with the method is making unbiased assumptions that produce fair comparisons of the alternate bids. The assumptions include project design life, material service life, project residual values at the end of its design life, recurring annual costs, rehabilitation costs and inflation and discount rates.

Design Life

Before any life cycle cost comparisons of materials can be made, the basic project design life must be established. In the case of some agencies it is already a matter of policy. For example, a 50 year design life for primary state highway culverts is common. The project design life has nothing directly to do with the various competitive materials available for the job. However, the life cycle cost analysis of competitive materials is directly affected by the project design life.

There are two key factors that determine a proper project design life. One is probable obsolescence (the longer the design life chosen the greater the risk of probable project obsolescence) and the other is available funds. A design engineer may ignore these factors and select a design life based only on his intuitive sense of logic. This mistake is particularly easy to make in the culvert and storm sewer field. Buried structures create a specter of excessive replacement costs; therefore, the tendency is to arbitrarily assign an excessive design life.

A rational determination of design life must consider obsolescence. How far in the future will the functional capacity be adequate? What adjacent development will take place? What future environmental regulations will require retrofit at the project site? What is required to increase the capacity? Is a parallel line feasible? Does location dictate destruction of the old pipe to build a larger structure? All these questions and others must be considered and evaluated. Do you oversize now or not? If so, how much? It may require life cycle cost analysis to evaluate the design capacity that is economically justified at this time to accommodate future requirements.

In addition to obsolescence in functional capacity, there is obsolescence in need. Will the basic facility be needed beyond some future date? The statistical probability that a specific facility will be totally abandoned after a certain period will set some upper limit of design life.

After rational study and economic analysis has determined a capacity (size), and a realistic design life for that capacity facility, there is still the question of available funds. Regardless of theoretical long-term economics, current resources will set practical limitations on building for future needs. Taxpayers and owners are not motivated to bear costs now that cannot possibly benefit them. This results in a limit on design life that could perhaps best be called political.

The result of these obsolescence and money factors is a practical limit on design life of 50 years for most public works projects. The taxpaying public can relate to a benefit to them in a 50 year life. Design lives exceeding 50 years are speculative at best.

Material Service Life

After the design life of the facility (sewer, culvert) has been selected, the service life of the alternative pipe materials must be established. The validity of the life cycle cost analysis will be no better than the estimated service life selected. Unless this selection is given adequate effort and an objective evaluation, the life cycle cost analysis will be only a mathematical exercise.

The average service life of various pipe materials varies with the environment, the effluent and the slope. Regional durability studies of culverts are available for most areas and can be used for storm drains too. Additionally, numerous published reports by agencies and organizations are available. In conjunction with simple jobsite tests of the environment and effluent, such reports can develop material service life appropriate for that region and application.

Refer to Chapter 9, Durability, for comprehensive guidance in determining service life for CSP.

Residual Values

Residual or salvage value reflects estimated economic value of the drainage facility at the end of project design life. While a used piece of construction equipment can be sold at auction at the end of its service life, drainage pipe—be it metal, concrete or plastic—is of little economic value. Often, projects to increase drainage capacity require that existing materials be removed before the end of their service life to permit expansion. The higher the likelihood of future *functional obsolescence,* the less likely there will be any salvage value. Concrete pipe proponents suggest that economic credit should be given when their estimated pipe service life exceeds the project design life (100 year pipe life vs 50 year project design life). Such calculations make it appear as if only one-half the cost of the pipe should apply to the project. This is inappropriate economic logic.

Recurring Annual Costs

All underground pipe systems require periodic inspection and maintenance. Typically, the costs for these preventative maintenance functions can be expected to occur in about the same amount (in constant dollars) from year to year. These costs need not be included in the study if they are expected to be the same for each pipe alternative. The present value (PV) for recurring annual costs can be calculated as:

$$PV = A_r = \frac{(1 + d)^{n-1}}{d\,(1 + d)^n}$$

Where: A_r = Recurring Annual Amount
d = Discount Rate
n = Number of Years

Rehabilitation vs. Replacement

The end of average service life does not mean replacement of the pipe as is often assumed in many life cycle articles. It does mean expenditure of funds at that time for pipe material maintenance. Planned maintenance always reduces the cost of "neglect and replace" practices. Inspections, even on only a 10 year frequency, will permit timely repair to be made while it is still inexpensive. The soundness and need for such inspections is essential to all infrastructures and must be done regardless of the materials involved. Such inspections allow low cost, planned maintenance. Actual rehabilitation cost will vary with the pipe size and the timeliness of the repair. This principle is entirely applicable to pipe culverts and storm sewers.

The normal type of rehabilitation required for a corrugated steel pipe line is invert repair. The typical pipe can be repaired and made serviceable for another "life cycle" with relatively modest invert treatment.

Based on prior and continuing technical advances, rehabilitation should be about 25% of original pipe cost. Higher costs would apply to rehabilitation of pipes not maintained at the end of their average service life. In those cases, however, many more years of service squeezed out of the structure offset some of that cost. For further information on pipe maintenance and rehabilitation see Chapter 12.

■ **Figure 11.5** There are many economical pipe rehabilitation techniques being used. One method employs the use of CSP to slip line distressed reinforced concrete pipe.

Discount Rates and Inflation

The method of handling these two economic values contributes to most of the confusion in developing life cycle cost comparisons. There are many articles and texts which debate whether to inflate or not, by how much, and what value to use for the discount rate. The logic for each seems coherent and yet, depending on the approach used, the calculations often result in completely different choices appearing to have the lowest cost. How can that be?

The answer lies in gaining an understanding of how the present value is affected over a range of discount rates. Present value is calculated as:

$$PV = A\left[\frac{1}{1+d}\right]^n$$

where A = Amount
d = Discount rate
n = Number of years until future expenditure occurs

In general, greater significance is given to future spending at low discount rates, and less significance at high discount rates, as shown in the following table:

Present Value of $1.00
Expended at Various Intervals and Discount Rates

	Discount Rate		
Year	**3%**	**6%**	**9%**
0	1.00	1.00	1.00
25	0.48	0.23	0.12
50	0.23	0.05	0.01
75	0.11	0.01	0.01

In contrast to the three times increase in discount rates from 3% to 9% there is a 23 times decrease in the significance in the present values of expenditures occurring in year 50 (.23 vs. .01). Also, since present value factors behave exponentially, a 3 point difference at higher rates (9% vs. 6%) has less present value significance than the same 3 point difference at low rates (3% vs. 6%).

Discount Rates

The discount rate is used to convert costs occurring at different times to equivalent costs at a common point in time. *The rate selected should reflect the owner's time value of money.* That is, the rate should represent the rate of interest that makes the owner financially indifferent between paying or receiving a dollar now or at some future time.

There is no single correct discount rate for all owners in either the public or private sector. Rate selection should be guided by the value of money to the owner. In the private sector, this is usually influenced by the rate of return the owner can achieve on projects that have comparable risk. This is sometimes referred to as the owner's "opportunity cost of capital."

In the public sector, discount rates are often mandated by policy or legislation. The Office of Management and Budget in Circular A-94 requires that federal projects use, in most cases, *a real discount rate* (net of inflation) of 7%.

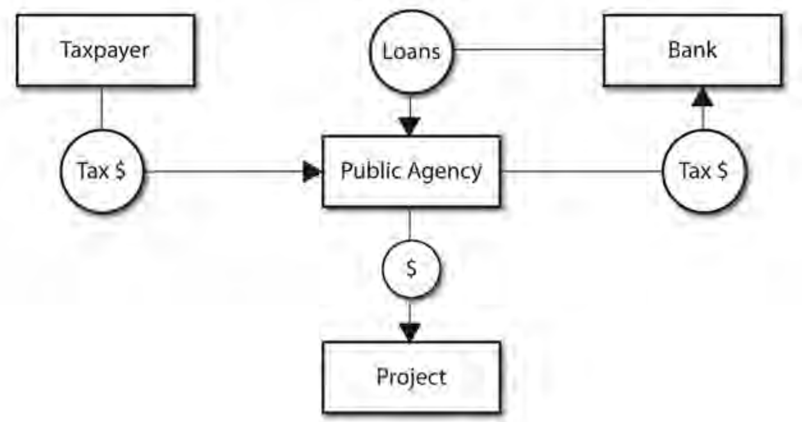
OMB recognizes that public investments displace both private capital and consumption. The use of a real discount rate of 7% …*approximates the marginal pretax rate of return on an average investment in the private sector…*" The U.S. Water Resources Council, Department of the Army and some, but not all, states have established guidelines and values for discount rates.

Borrowing Rates

There is a tendency in the public sector to base the discount rate on the cost to borrow money (interest rate on bonds). This is incorrect. The interest rate on bond financing represents a cost to the project and does not reflect the value of money used on the projects. The distinction between cost and value is subtle but important.

Borrowed money does not pay for the project, taxpayers do. Borrowed funds are repaid, over time, with taxes collected from taxpayers. Therefore, the discount rates used for public projects should be based on the time value of money to the taxpayer, which will always be greater than the interest rate on public bonds.

The following diagram shows the financial relationship between taxpayers, public agencies and bank borrowing.



In the end, taxpayers pay for public projects. Therefore, it is never appropriate to use the interest rate on borrowed money for the discount rate. A common sense test of any proposed discount rate is whether you would want your pension to be invested at that rate. In that perspective the 7% (real) discount rate prescribed in A-94 is realistic.

Inflation

Several approaches can be used in the treatment of inflation. First, the analyst should determine whether any legislated or mandated policy applies to the project under consideration. If not, then a straight forward approach can be used. All costs, both present and future, can be estimated in base year or current year dollars and discounted back to the present using a "real" discount rate (net of inflation). This approach is the most commonly used and eliminates the complications that are associated with making future projections of inflation.

The real discount rate (d_r) and its corresponding nominal discount rate (d_n) are related as follows:

$$d_r = \frac{1 + d_n}{(1 + I)} - 1 \quad \text{or} \quad d_n = (1 + d_r)\,(1 + I) - 1$$

where I = the general rate of inflation.

The real discount rate can be calculated based on a user selected nominal discount rate and general rate of inflation. For example, a 10% nominal discount rate and a 3% inflation rate results in a real discount rate of 6.8% (Note: This is slightly different result than the arithmetic difference between 10% and 3%).

A less direct approach, but one yielding the same results, is for the analyst to make specific projections of future costs. Future costs can be projected by multiplying the estimated cost expressed in base year or current cost dollars by the inflation factor $(1+I)^n$ where I is the general rate of inflation and n is the number of years into the future.

A third method is to apply inflation selectively to certain elements of cost. For example, some federal agencies are required to recognize inflation on energy costs only; general inflation is to be ignored. Dealing with inflation incrementally adds to the computational complexity. Those interested in this approach should consult TM 5-802-1 listed in the bibliography for practical application of this technique.

Recommendations

The analyst must first determine if the project owner has or is subject to any policy that specifies the treatment of discount rates and inflation. In the absence of specific guidance, it is recommended, consistent with OMB A94, that a *real* discount rate of 7% be used and all costs estimated in current period dollars. If a requirement exists to recognize inflation, then use a *nominal* discount rate of 10% and a long term inflation rate of no more than 3%.

Calculations

The following example is presented to illustrate the comparison on two drainage pipe alternatives. Results are based on calculations carried to the fifth decimal, rounded as shown

- Basic Assumptions
  - — Project Design Life: 50years
  - — Owner Selected
    - Discount Rate (d_n) 10% (nominal)
    - Inflation Rate (I): 3%
- Corrugated Steel Pipe
  - — Initial Cost: $150,000
  - — Service Life: 40 Years
  - — Current Cost of Invert Rehabilitation at 25% of Initial Cost: $37,500
  - — Salvage Value: None
  - — Annual Maintenance Cost: $500
- Concrete Pipe
  - — Initial Cost: $180,000
  - — Service Life: 60 Years
  - — Salvage Value: None
  - — Annual Maintenance Cost: $500

Since the $500 annual maintenance costs affect both cases equally, they can be excluded from the analysis. The next step is to calculate the real discount rate where:

$$d_r = \frac{1 + d_n}{1 + I} - 1$$

$$= \frac{1.10}{1.03} - 1 = \mathbf{.068 \text{ or } 6.8\% \text{ real discount rate}}$$

The present value for the CSP alternative is then determined as:

Initial Cost	$150,000
Rehabilitation Cost	
$37,500 x 0.0721* =	2,703
Total Present Value	$152,703

$$* = \frac{1}{(1 + .068)^{40}} = 0.0721$$

Since the concrete pipe alternative is estimated not to require future expenditures, its present value is equal to its original cost of $ 180,000. Accordingly, CSP has a lower present value and therefore, represents the lower cost alternative.

	Present Value
Concrete Pipe	$180,000
Corrugated Steel Pipe	152,703
CSP Advantage	$ 27,297

Sensitivity of Assumptions

A sensitivity analysis can be used to determine how variations in key assumptions affect the outcome of the life cycle cost analysis. This can be particularly helpful if the present values of alternatives are close or there is uncertainty regarding certain assumptions.

In general, the two factors having the greatest influence on the ranking of alternatives are the magnitude of the discount rate and the differential in initial costs. The significance of future expenditures is lessened at higher discount rates and increased at lower discount rates. Reasonable variations in the magnitude and timing of future expenditures usually have only a small effect on the results. Based on the proceeding example, the following table illustrates how reasonable variations in assumptions affect the $27,297 difference in present value.

Basic Assumption	Variation	Approximate Increase/ (Decrease) in $27,297 Present Value Differential
6.8% Real Discount Rate	4.8% 8.8%	$(3,000) 1,400
Rehabilitate in 40 Years	35 years 45 years	(1,000) 800
25% rehabilitation cost	20% 50%	500 (2,700)

Computer Program

The National Corrugated Steel Pipe Association has developed a computer spreadsheet template designed to evaluate up to three alternatives simultaneously. User selected input can be easily modified to perform sensitivity analysis. Output can be reviewed on screen or printed. To obtain the program, visit the NCSPA website at www.NCSPA.org.

Summary

The principles of value engineering are essential in a cost-effective approach to design. Life cycle cost is an especially effective method to compare alternatives that are characterized by different cash flows over the project life. The method requires objective and realistic assumptions concerning project design life, material service life, residual values, future expenditures, the owner's time value of money (discount rate) and future inflation.

Nested and stacked CSP.

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Rehabilitation & extended concrete box.

CHAPTER twelve

Inspection, Maintenance and Rehabilitation

INSPECTION

With an aging infrastructure, and a vast number of pipes and culverts in the transportation system, deterioration is a growing problem for transportation agencies. The traveling public does not see most pipes and many are located on roads with high volumes of traffic and often under high embankments. Even with scheduled bridge inspections that include the larger drainage structures, the condition of these pipes often becomes noticeable only after a problem arises such as settlement of the road, pipe failure or flooding. Once a problem arises, the cost of replacement not only includes the direct construction costs but also significant indirect costs as a result of accidents, delays, detours, cleanup due to pollutants leaking into the groundwater etc.

Drainage systems should be inspected on a routine basis to ensure they are functioning properly. Depending on pipe size, flow area, risk associated with pipe location, Average Daily Traffic, detour length, previous inspection rating, etc., structures may need to be inspected annually or on a two, three or four-year cycle. Inspections should also always be conducted following a major storm. Systems that incorporate infiltration are most critical since poor maintenance practices can soon render them inefficient. An efficient pipe assessment and maintenance program will aid in reducing failures and in the cost-effective planning and prioritizing of future replacement, repair or rehabilitation of the drainage structures.

Thorough inspection of sewers, culverts and other soil-metal structures should be conducted using established guidelines under which all inspectors follow and record findings in the same manner. Inspection can be carried out visually and recorded on standard forms as well as with still photographs or videos. Current electronic technology enables the inspector to communicate directly with a central office using facsimile or video transmission. Inspection software has also been developed by various highway departments which provide a more consistent method of reporting. The date of installation, a description and configuration of the product, the date of subsequent inspections and maintenance, should be properly recorded. The FHWA Culvert Inspection Manual (1986), AASHTO Highway Drainage Guidelines, Volume XIV and NCHRP Synthesis 303 all provide guidelines for inventory, inspection and evaluation of existing culverts. The Transportation Research Record published a culvert ranking model based on economic factors in 1991. More recently, numerous researchers and highway departments have developed guidelines on culvert inspection, rating and management as well as culvert cleaning and repair practices (California, Kansas, Maine, Missouri, Montana, New Jersey, Ohio, Tennessee, and Utah).

Inspection requires both environmental and structural assessment. These are discussed in the following section.

■ **Figure 12.1** Pipe inspection.

ENVIRONMENTAL ASSESSMENT

Environmental assessment includes the conditions of soil side corrosion, water side corrosion, water side abrasion and clogging. Soil side corrosion can be determined by coring the structure and evaluating the soil-side coupon. Inspection includes a visual examination for spalling, red rusting, pitting and perforating. The soil corrosivity, including pH and resistivity, is recorded along with moisture, soluble salts and oxygen content.

Water side corrosion is determined by a visual examination for spalling, red rusting, pitting and perforating. The water corrosivity including pH, resistivity and hardness is also recorded.

Water side abrasion is also determined visually along with an assessment of structure slope, flow velocity and upstream bed load of either rock or sand.

Clogging due to the accumulation of sediment or debris can be readily assessed visually and measured manually.

Other environmental factors that could influence the service life of the culvert should also be noted. These could include such factors as anticipated changes in the watershed upstream of the culvert, industrial effluents, stray electrical currents and possible effects of severe climates.

See Chapter 9, Durability, for guidelines on corrosiveness of soils and water, abrasion, and information on site testing equipment.

STRUCTURAL ASSESSMENT

Structural assessment includes shape monitoring and investigating for signs of joint separation, crimping of the pipe wall, excessive deformation, invert lifting, pipe end lifting and pipe end distortion. Additional structural assessment requirements for structural plate soil-steel structures include inspecting for bolt hole tears and structural distress at the longitudinal seams. The National Corrugated Steel Pipe Association (NCSPA) offers a rating methodology for structural evaluation of in-service corrugated steel structures. It is listed as NCSPA Design Data Sheet No. 19 and is available from the NCSPA or member companies.

Shape Monitoring

Traditional monitoring methods have usually consisted of a visual inspection with measurements being taken only if serious signs of distress are observed. Other than the span and rise, geometric measurements are normally limited to selected chord lengths and offsets at specified cross sections. These dimensions can be related to the curvature of a section. The performance of the structure is judged primarily on deformation stability. Excessive flattening of a section makes it susceptible to snap-through instability. The extent of flattening that a structure can tolerate is not easily defined. Arbitrary limits on the reduction in the mid-ordinate heights have been used to define the severity of the deformations and the remedial measures required. The changes are usually measured from the design shape, since the as-built dimensions are rarely measured. Being flexible, it is possible that the pipe was deformed from the design shape during construction, with little or no subsequent deformations. Only an ongoing monitoring of the structure can confirm that the deformations have stabilized.

In photogrammetric monitoring, an object is photographed using specialized equipment following set procedures, and measurements are obtained from the photographic images. These measurements and some externally supplied information are used to determine, either analogically or analytically, the location of reference points in the three-dimensional object space. Photogrammetry is particularly useful in monitoring large or difficult-to-access structures.

Crimping of Pipe Wall

Crimping can be regarded as a consequence of local buckling in which the metallic shell buckles into a large number of waves, each of relatively small length. It can occur in the compression zone of the wall section when the pipe wall undergoes large bending deformations. This kind of crimping usually takes place in pipe wall segments of relatively small radius. It indicates that the soil behind the segment is not dense enough to prevent excessive bending deformations.

Crimping can also occur in an entire pipe wall section subjected to excessive thrust while being supported by very well compacted backfill. Although the incidence of this kind of crimping is rare, it is known to have occurred in structures with circular pipe, which were constructed with good-quality, well-compacted backfill on a relatively yielding foundation. It is assumed that the long-term foundation settlements of these structures induced negative arching, thus subjecting the pipe wall to greater and greater thrusts as time passed, until the thrust exceeded the buckling capacity of the pipe wall even though it was well supported horizontally.

Buckling of the entire pipe wall section into waves of small length has a redeeming feature. By reducing the axial rigidity and increasing axial deformations of the pipe, it induces positive arching, thus effectively reducing the axial thrust in the pipe. The result of this sequence is that, despite crimping, the pipe can be in a stable condition provided, of course, that the time-dependent foundation settlements have ceased.

If the only sign of distress in a soil-steel structure is crimping limited to a few segments, then in most cases one need not be too concerned about the structural integrity.

Excessive Deformation

Excessive deformations of the pipe wall are caused by the inability of the backfill to restrain its movement. Because of its flexibility, the pipe can deform excessively during the initial stages of the backfilling operation. If such deformation is not prevented or corrected during construction, the structure is built with deformed pipe. Pipe deformations locked in during construction may not be detrimental to the structural integrity of the structure, especially if they have stabilized. Pipe deformation occurring after the completion of the structure may, on the other hand, be a warning signal for the imminent collapse of the structure.

It is very important that a record of the as-built pipe shape be kept so that it can be ascertained later whether the observed deformation occurred recently or has been there since the construction of the structure. When the records of the as-built structure are not available, it is important to record the changes in the pipe shape at regular intervals after the deformations were first noticed. If the deformations are not significant and have not undergone significant changes, then it is likely that the structure will perform as intended.

Lifting of Invert

In soil-steel structures with a large radius or flat invert plates, the pressure under the corners is much greater than under the invert plates. If the foundation has inadequate bearing capacity the structure settles more under the haunches than under the invert. This results in a loss of waterway area and may deteriorate the structural capacity of the structure.

Lifting of Pipe Ends

This is another form of distress sometimes found in hydraulic structures. This kind of distress is caused by a combination of uneven settlement of the pipe foundation along its length, buoyancy effects and CSP joints located too close to the end of the pipe.

Distortion of Beveled Ends

Beveled ends are particularly vulnerable to damage by horizontal pressures. A complete pipe provides a closed section and thus, can sustain much higher intensities of lateral pressure than an incomplete ring. Lacking a closed section, the beveled ends of a pipe are prone to damage by lateral earth pressures or from vehicle impact or heavy equipment pieces falling on them. To resist this, beveled ends can be reinforced or tied back to deadman anchors in the fill.

Joint Separation

CSP offers some of the structurally strongest joints available, yet separation of the joints may occur due to uneven settlement along the pipe, steep slopes, improper bedding, backfill or alignment during construction or movements due to earthquakes or frost.

Bolt-hole Tears in Structural Plate Structures

Bolt-hole tears or cracks may occur in longitudinal seams. Since the pipe wall is always subjected to compressive forces, the bolt-hole tears usually do not extend over the entire section of the wall.

Bolt-hole tears are most common in pipe arches at the longitudinal seam between the top and side segments of the wall. This location predominates because excessive shape changes induce excessive bending and bolt tension at the inside valley bolt holes. However, they are also found, very infrequently, in other soil-steel structures. Bolt-hole tears are not always the result of excessive deformation of the pipe wall of the completed structure; they can also be formed during assembly when poorly matching plates are forced to fit at the longitudinal seams.

Reviews of this issue have been conducted and all reports conclude that there is a correct and incorrect way of lapping the plates at longitudinal seams.

The correct orientation for longitudinal seam plate lap results in the valley bolt being located closest to the visible plate edge. This is illustrated in Figure 12.2.

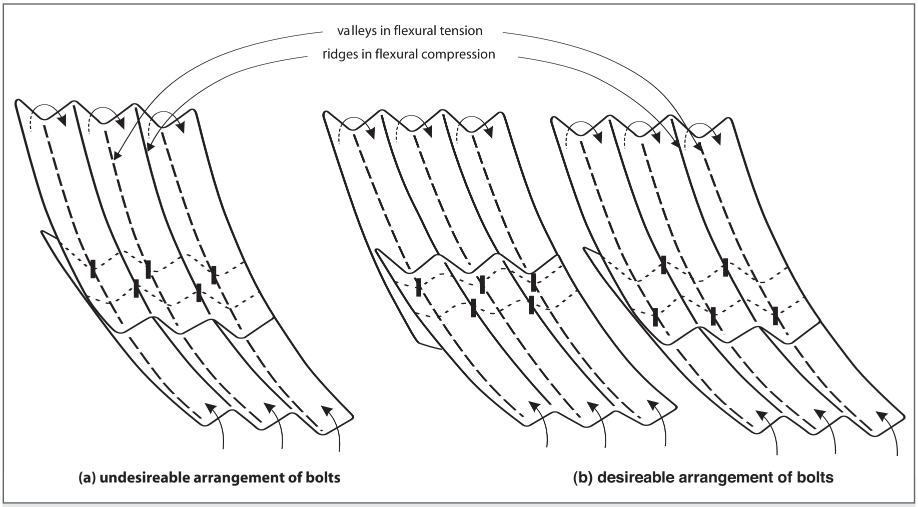


Figure 12.2 (a) Undesirable and (b) desirable arrangement of bolts. (Reproduced from Canadian Journal of Civil Engineering, Volume 15, Number 4, 1988, Pages 587 - 595: Bakht and Agarwal)

Distress at Longitudinal Seams in Structural Plate Structures

Distress at longitudinal seams can be observed as a result of yielding of the wall directly under the bolts or shear failure of the bolts. Such distress occurs under excessive pipe wall thrust and under conditions that preclude excessive bending deformations.

While failure of longitudinal seams has been observed in laboratory testing to determine strength of bolted joints, it is extremely rare to find in practice.

MAINTENANCE

Procedures and equipment for maintenance of drainage systems are discussed in this section. As with inspection, good records should be kept on all maintenance operations to help plan future work and identify facilities requiring attention. Various types of equipment are available commercially for maintenance of drainage systems. The mobility of such equipment varies with the particular application and the equipment versatility.

Trenches

The clogging mechanism of trenches is similar to that associated with other infiltration systems. Although the clogging of trenches due to silt and suspended material is more critical than that of basins, it is less critical than the clogging of vertical wells. The use of perforated pipe will minimize clogging by providing catchment for sediment without reducing overall efficiency. Maintenance methods associated with these systems are discussed later in this chapter.

Catch Basins

Catch basins should be inspected after major storms and cleaned as often as needed. Various techniques and equipment are available for maintenance of catch basins as discussed in the next section. Filter bags can be used at street grade to reduce the frequency for cleaning catch basins and outflow lines.

Vacuum Pump

This device is normally used to remove sediment from sumps and pipes and is generally mounted on a vehicle. It usually requires a 200 to 300 gal. holding tank and a vacuum pump that has a 10 inch diameter flexible hose with a serrated metal end for breaking up caked sediment. A two-man crew can clean a catch basin in 5 to 10 minutes. This system can remove stones bricks, leaves, litter, and sediment deposits. Normal working depth is 0 to 20 feet.

Waterjet Spray

This equipment is generally mounted on a self-contained vehicle with a high pressure pump and a 200 to 300 gal. water supply. A three inch diameter flexible hose line with a metal nozzle that directs jets of water out in front is used to loosen debris in pipes or trenches. The nozzle can also emit umbrella-like jets of water at a reverse angle, which propels the nozzle forward as well as blasting debris backwards toward the catch basin. As the hose line is reeled in, the jetting action forces all debris to the catch basin where it is removed by the vacuum pump equipment. The normal length of hose is approximately 200 feet. Because of the energy supplied from the waterjet, this method should not be used to clean trench walls that are subject to erosion.

Bucket Line

Bucket lines are used to remove sediment and debris from large pipes or trenches (over 48 inch diameter or width). This equipment is the most commonly available type. The machine employs a gasoline engine driven winch drum, capable of holding 1000 feet of $^1/_2$ inch diameter wire cable. A clutch and transmission assembly permits the drum to revolve in a forward or reverse direction, or to run free. The bucket is elongated, with a clam shell type bottom that opens to allow the material to be dumped after removal.

Buckets of various sizes are available. The machines are trailer-mounted usually with three wheels, and are moved in tandem from site to site. When a length of pipe or trench is to be cleaned, two machines are used. The machines are set up over adjacent manholes. The bucket is secured to the cable from each machine and is pulled back and forth through the section until the system is clean. Generally, the bucket travels in the direction of the flow and every time the bucket comes to the downstream manhole, it is brought to the surface and emptied.

Fire Hose Flushing

This equipment consists of various fittings that can be placed on the end of a fire hose such as rotating nozzles, rotating cutters, etc. When this equipment is dragged through a pipe, it can be effective in removing light material from walls. Water can be supplied from either a hydrant or a truck.

Sewer Jet Flushers

The machine is typically truck-mounted and consists of a large water tank of at least 1000 gal., a triple action water pump capable of producing 1000 psi or more pressure, a gasoline motor to run the pump, a hose reel large enough for 500 feet of 1 inch inside diameter high pressure hose and a hydraulic pump to remove the loose material. In order to clean pipes properly, a minimum nozzle pressure of 600 psi is usually required. All material is flushed ahead of the nozzle by spray action. This extremely mobile machine can be used for cleaning areas with light grease problems, sand and gravel infiltration, and for general cleaning.

REHABILITATION

Rehabilitation of the infrastructure is a major undertaking now being addressed by federal, state, and local governments. While the magnitude of rehabilitation may at times appear enormous, rehabilitation often is very cost-effective when compared to the alternative of new construction.

This section deals primarily with the use of CSP, corrugated structural plate, or steel tunnel liner plates to cost-effectively restore the structural capacity of deteriorated or failing culvert and bridge structures. These deteriorated structures include precast or cast-in-place concrete structures, concrete, steel or plastic pipe and all types of existing bridge structures. The corrugated steel material is typically inserted or assembled inside the failing structure and the annular space between the liner and deteriorated structure is filled with grout. This rehabilitation technique is commonly referred to as slip lining. After the grout has set, the repaired structure usually becomes much stronger than the original one and remains virtually free of distress.

Deteriorated CSP structures can often be rehabilitated by merely providing a new wear surface in the invert. If necessary they can also be repaired by slip lining or by a number of other methods to provide a new, complete service life at a fraction of the cost or inconvenience of replacement. A practice for placing a concrete invert or entire lining is provided in ASTM specification A 979/A 979M. Some of these repair and rehabilitation methods are also described in this section.

All of the methods described herein require a complete inspection and evaluation of the existing pipe to determine the best choice of corrugated steel material and coatings.

Rehabilitation by Slip Lining

Materials and Details for Slip Lining

Use of CSP or structural plate products has a number of advantages for sliplining applications. Standard corrugated steel pipe, manufactured in accordance with AASHTO M 36 or ASTM A 760/A 760M, may be provided in any lengths which would facilitate insertion at the site. The CSP liner pipe can be manufactured to any standard size or virtually any custom size, round or arched in shape, to fit the existing pipe cross-section. Accurate surveying of the existing structure is important to determine the maximum size of liner pipe that can be installed. A hydraulic advantage may be gained by using helical corrugated steel pipe if the existing pipe is annular corrugated or hydraulically rough due to excessive deformations. If the owner desires to maintain maximum hydraulic capacity of the line, then the use of a smooth lined corrugated steel pipe is recommended. Choices of this type of pipe include ribbed pipe, double wall CSP, 100% cement mortar lined, and 100% asphalt lined. A number of coatings are also available to provide the required remaining design life for the structure (see Durability Chapter 9).

Figure 12.3 Typical slip lining installation with a pipe arch shape to suit existing cross-section.

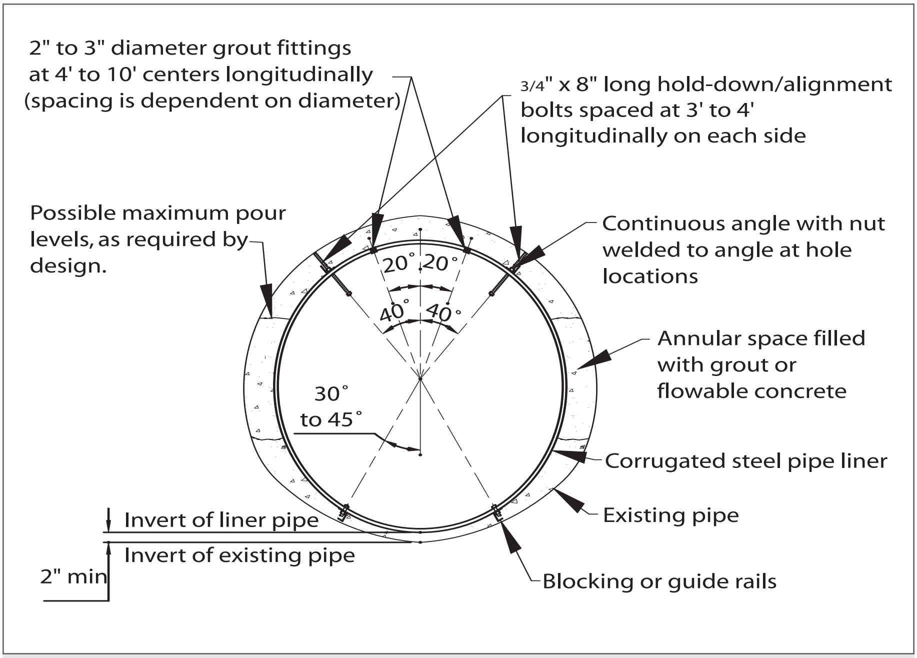
CSP is lightweight, making it easy to handle and install. Skid devices may consist of steel or pressure treated wood guide rails placed on the invert of the existing pipe, or skid bars attached to the liner pipe to facilitate moving it in to the desired location. These devices also maintain a minimum space between the bottom of the liner and the existing structure to facilitate proper grouting. Adjusting bolts are used to secure the slipline pipe in position and prevent floating of the pipe during grouting. These usually consist of 3/4 inch diameter rods, threaded through nuts or steel angles welded to the outside of the liner plate. Typical spacing is approximately 5 to 10 foot centers and at about 40 degrees on each side of the top centerline. For relining a concrete box shape, three sets of these rods would be used, located at the top and spring line on each side of the liner pipe. Grout fittings are welded to the liner pipe wall at positions and spacing as determined by the Engineer. See Figures 12.8 and 12.9 for typical details of slip line installations.

■ **Figure 12.4** Pipe sections can be fabricated in lengths as required to suit slip lining installation.

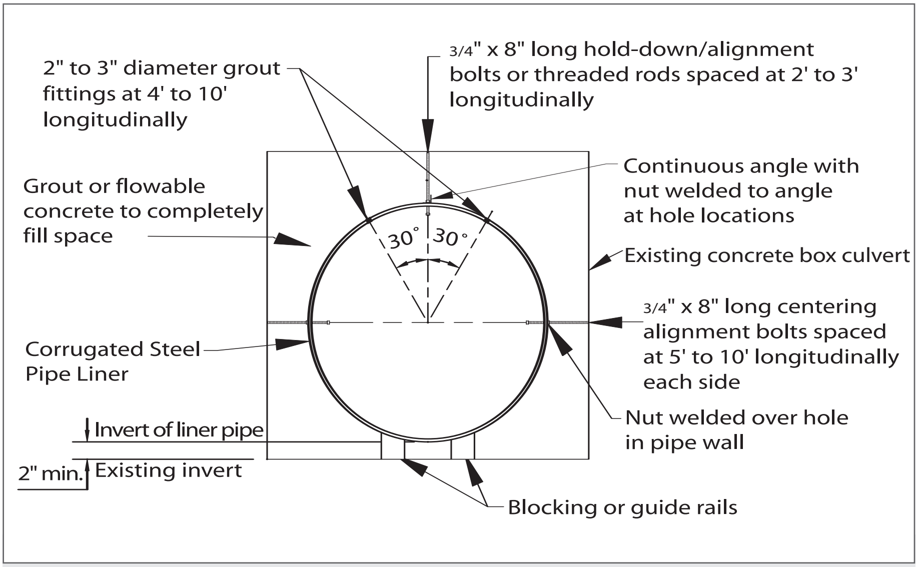
■ **Figure 12.5** Liner pipe is easily installed by pushing or pulling through host pipe using conventional equipment.

■ **Figure 12.6** Liner pipe being installed in an existing concrete box.

■ **Figure 12.7** Deep corrugated box culvert lining a falling bridge. Note the dozer pulling the corrugated box culvert into position.

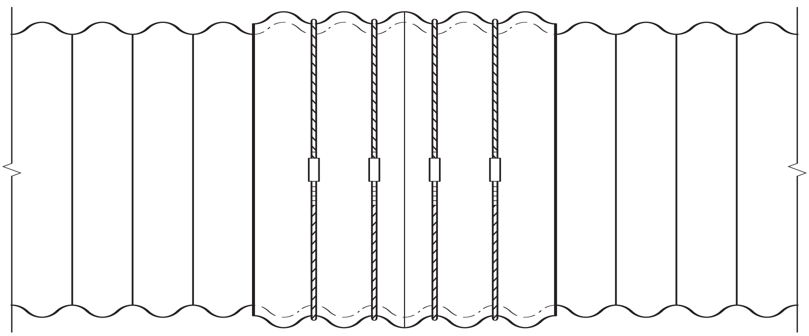


■ **Figure 12.8** A typical section of a corrugated steel pipe slip line installation in a round pipe.

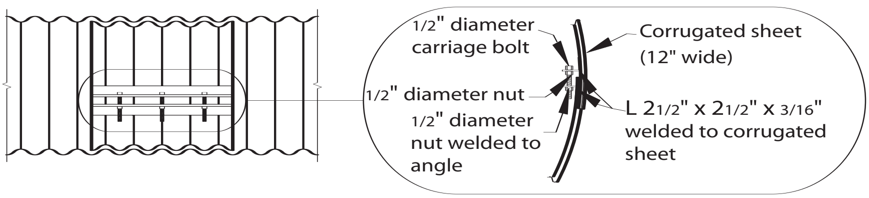


■ **Figure 12.9** A typical section of a corrugated steel pipe slip line installation in a concrete box culvert.

Various types of connecting bands and gaskets are available to suit site requirements. If sufficient clearance exists between the liner pipe and the existing line, the sections may be joined by the use of a threaded rod and lug type coupling band as shown in Figure 12.10. Internal expanding coupler bands as shown in Figure 12.11 are recommended if there is insufficient clearance on the outside of the liner pipe. Another alternative is to use sheet metal screws or Huck© rivets in conjunction with an installation jig.



■ **Figure 12.10** Band is secured by rods around band connected by lugs.



■ **Figure 12.11** The use of an internal expanding type coupling band is recommended to connect the section if there is insufficient clearance on the outside of the liner pipe.

For large deteriorated structures and bridges, corrugated structural plate or tunnel liner plate can be used as the liner, either assembled in-place or outside the structure and slid into place as with CSP. Structural plate can be manufactured in a variety of shapes to suit the project including round, pipe arch, arch, box shape and elliptical. Deep and shallow corrugated plate is available, dependent on the design requirements. It is important to work with the pipe manufacturer and the contractor to determine the material and details for connection and installation to best suit the project.

■ **Figure 12.12** Lining a concrete box with a steel box culvert.

Grouting of Slip Lined Pipe

Bulkheads must be placed at each end of the installation or at staged intervals for longer projects. Grouting of the annular space is usually accomplished using a sand-cement grout mixture, progressively pumped at low pressure along the length of the pipe using spaced grout fittings starting at the low end. Depending on the length of the liner, the grout can sometimes be pumped through an opening in the end bulkheads. The grout pressure and flow rate must be carefully controlled to prevent failure of the liner pipe and excessive flotation forces. The grout is placed in lifts to limit flotation and buckling pressures. It is important to follow a progressive grouting procedure, utilizing inspection ports, to completely fill the annual space. In some installations the grout is placed through an opening at the roadway surface above the liner pipe.

The grout material should be a non-shrink mix with a minimum 28-day compressive strength of 300 psi. Non-corrosive fly ash and non-chloride admixtures are generally permitted in the grout mix. A typical grout mixture could consist of one part of cement and five parts fine aggregate, by volume, with 10 pounds of fly ash added for each bag of cement. If internal coupling bands are used they can be removed after the grout has achieved initial set.

Figure 12.13 Structural plate arch with grout plugs used to line concrete box culvert.

Figure 12.14 Grout can be pumped into annular space through an opening in the end bulkheads.

■ **Figure 12.15** Finished end treatment.

Design of Slip Line Pipe

The liner pipe is typically designed as a conventional flexible structure to carry all the imposed dead and live loads. However, if grouting is controlled or appropriate blocking techniques are used, the flexibility requirements can be relaxed. The existing structure, which will remain in place and continue to support all or a portion of the loads on it, and the contribution of the surrounding grout are generally totally ignored. A new analysis method for the complex soil-structure interaction of the grouted system has not yet been developed and verified. The liner installation must also be checked for buckling due to anticipated hydrostatic or grout pressures and flotation forces for the installation.

Rehabilitation of CSP and Structural Plate Pipes

Although a pipe may be deemed structurally deficient, this does not rule out rehabilitation. In fact, an existing structure that has not failed still has a factor of safety of 1.0 or more. Repair methods can be utilized and the structure restored to structural adequacy and then normal rehabilitation procedures performed. Even with 25% metal loss, which occurs long after first perforation, structural factors of safety are reduced by only 25%. When originally built, CSP designs often provide factors of safety of 4 to 8—far in excess of that required for prudent design. This section deals mainly with the rehabilitation or repair of corrugated steel pipe and/or steel structural plate.

There are two categories of pipe rehabilitation, invert rehabilitation and total pipe rehabilitation.

Invert Rehabilitation

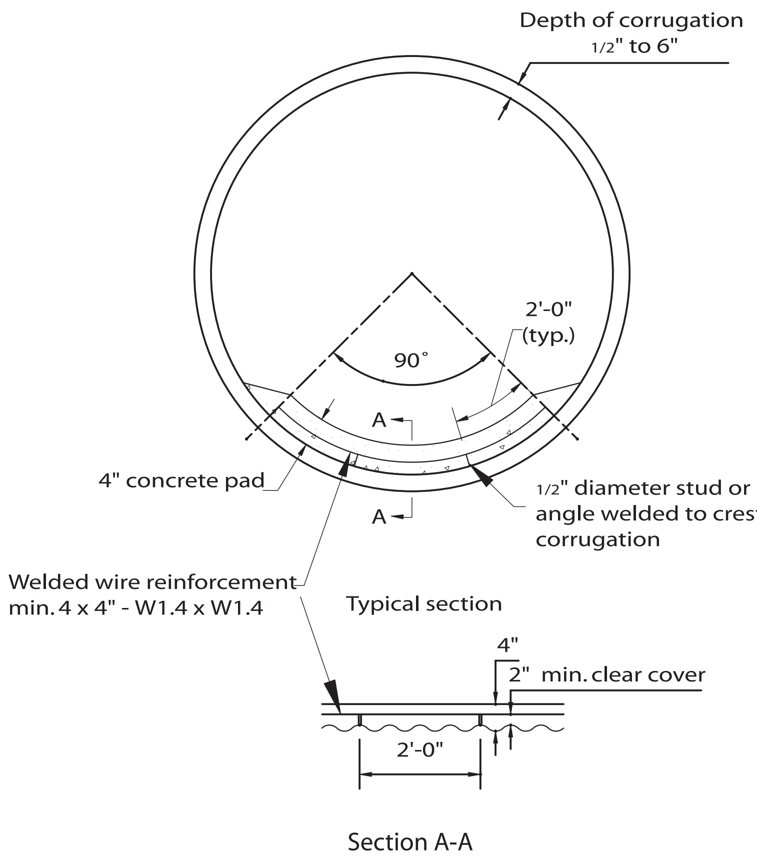
The inverts of CSP and structural plate pipes usually deteriorate faster than the rest of the culvert. When deterioration is limited to the invert, the pipe can be rehabilitated by replating the invert with new corrugated steel plates or by attaching heavy gage flat steel armor plates to the sides and bottom of the invert, Figure 12.16. For larger diameters where it is possible for a person to enter the pipe, a new concrete invert, or pavement, may be cast in the pipe. Plain troweled concrete may be satisfactory for mild conditions of abrasion and flow. For more severe conditions a reinforced concrete pavement is required. Figure 12.17 shows typical reinforcing and configuration of field placed concrete pavement. The final design should be in the control of the Engineer and depends upon the extent of the deterioration of the pipe. Additional details and specifications are provided in ASTM specification A 979/A979M.

Figure 12.16 Steel armor-plated invert.

Total Pipe Rehabilitation Using Relining Materials

The selection of the reline material is dependent upon the conditions of the pipe line to be rehabilitated and the diameter and/or shape. If the line has deteriorated to the point where it is deficient structurally, the choice would necessarily have to be a material having full barrel cross section with sufficient structural capability to withstand the imposed dead and live loads.

The following is a discussion of reline materials and methods of installing them. It is the Engineer's responsibility to select the material and method of relining dependent upon rehabilitation requirements. Alternate types of materials may be found in ASTM A 849.



■ **Figure 12.17** In-place installation of concrete invert.

Slip Lining

Slip lining a deteriorated pipe using CSP is usually the most cost-effective means of restoring the full structural capacity of the pipe where some downsizing of the line is not a concern. Often the original pipes are oversized to allow for future sliplining with a size meeting the hydraulic requirements. Inlet improvements may also compensate for the reduced flow area. The materials, procedures and details for slip lining CSP are as described previously for rehabilitating other types of deficient structures.

Figure 12.18 Concrete paved invert in structural plate pipe.

Figure 12.19 Slip lining with CSP.

Inversion Lining

Inversion lining is accomplished by using needle felt, of polyester fiber, which serves as the "form" for the liner. The liner expands to fit the existing pipe geometry and therefore is applicable to round, elliptical pipe and pipe arch. The use of this method requires that the pipe be taken out of service during the rehabilitation period. One side of the felt is coated with a polyurethane membrane and the other is impregnated with a thermosetting resin.

The felt variables include denier, density, type of material, method of manufacture (straight or cross lap), and length of fiber. The physical properties of the felt and chemicals must be determined for the specific project and in cooperation with prospective contractors.

Inversion lining has been utilized on lines from 4 to 108 inches in diameter. It is normally applicable for distances of less than 200 feet or where ground water, soil condition, and existing structures make open excavation hazardous or extremely costly. Inversion lining with water is generally confined to pipelines with diameters less than 60 inches and distances less than 1000 feet. Normally air pressure is utilized for inversion technique on larger diameter pipe. Compared with other methods, this process is highly technical. Other technical aspects include resin requirements which vary with viscosity, felt liner, ambient temperatures, and the filler in the felt content; the effects of ultraviolet light on the resin and catalyst; and safety precautions for personnel and property.

Shotcrete Lining

Shotcrete is a term used to designate pneumatically applied cement plaster or concrete. A gun operated by compressed air is used to apply the cement mixture. The water is added to the dry materials as it passes through the nozzle of the gun. The quantity of water is controlled within certain limits by a valve at the nozzle. Low water ratios are required under ordinary conditions. The cement and aggregate is machine or hand mixed and is then passed through a sieve to remove lumps too large for the gun.

When properly made and applied, shotcrete is extremely strong, dense concrete, and resistant to weathering and chemical attack. Compared with hand placed mortar, shotcrete of equivalent aggregate-cement proportions usually is stronger because it permits placement with lower water-to-cement ratios. For relining existing structures, the shotcrete should be from 2 to 4 inches thick depending on conditions and may not need to be steel reinforced. If used, the cross-sectional area of reinforcement should be at least 0.4% of the area of the lining in each direction.

Shotcreting with a steel-fiber-reinforced concrete mix has been used successfully to line the inside of soil-steel bridges in distress. The lining, which is up to 6 inches thick, may cover the complete perimeter of the cross section of the pipe. Alternatively, it may be limited to the damaged zone of the pipe wall.

When shotcreting is used for the complete ring, shear connectors are not usually provided between the pipe wall and the shotcrete. However, their inclusion can certainly increase the strength and stiffness of the additional ring.

The partial shotcrete ring is provided to repair localized damage, such as a section containing bolt-hole tears. For the partial ring, it is important to provide a shear connection between the pipe and shotcrete. This shear connection may be by shear studs of the type used in composite beams, or machine-welded to the pipe after the zinc coating from the galvanized plate has been ground off locally. An alternative to the usual shear stud is a U-shaped bracket of 12 gage galvanized steel sheet, which is attached to the pipe through pins, fired by a ram-setting gun. This type of shear connector, which is shown in Figure 12.20, has been used successfully in several cases.

Despite the ability of the fiber-reinforced concrete to sustain fairly large tensile stresses, it is advisable to add a steel reinforcement mesh to the shotcrete ring, especially if it is partial. Repair by fiber-reinforced shotcrete can prove economical and effective in many cases, mainly because of the fact that it requires no formwork and little preparatory work. Because of its relatively thin layer, the shotcrete ring does not reduce the pipe size appreciably. The repair work by shotcreting can be undertaken even in cold weather, provided that the pipe wall sections to be shotcreted are adequately heated. If only the top and side segments are to be repaired, then shotcreting of culverts can be carried out without diverting the stream.

The following specifications should be considered:

1. "Specifications for Concrete Aggregates", ASTM C 33.
2. "Specifications for Materials, Proportioning and Application of Shotcrete", ACI 506.
3. "Specifications for Chemical Admixtures for Concrete", ASTM C 494.

Cement Mortar Lining

Cement mortar lining is particularly well suited to small diameter pipe that is not easily accessible.

The cement mortar lining is applied in such a manner as to obtain a 1/2 inch minimum thickness over the top of the corrugations. Application operations should be performed in an uninterrupted manner. The most common practice uses a centrifugal machine capable of projecting the mortar against the wall of the pipe without rebound, but with sufficient velocity to cause the mortar to be densely packed in-place. See Figure 12.21, which shows a typical setup for this process.

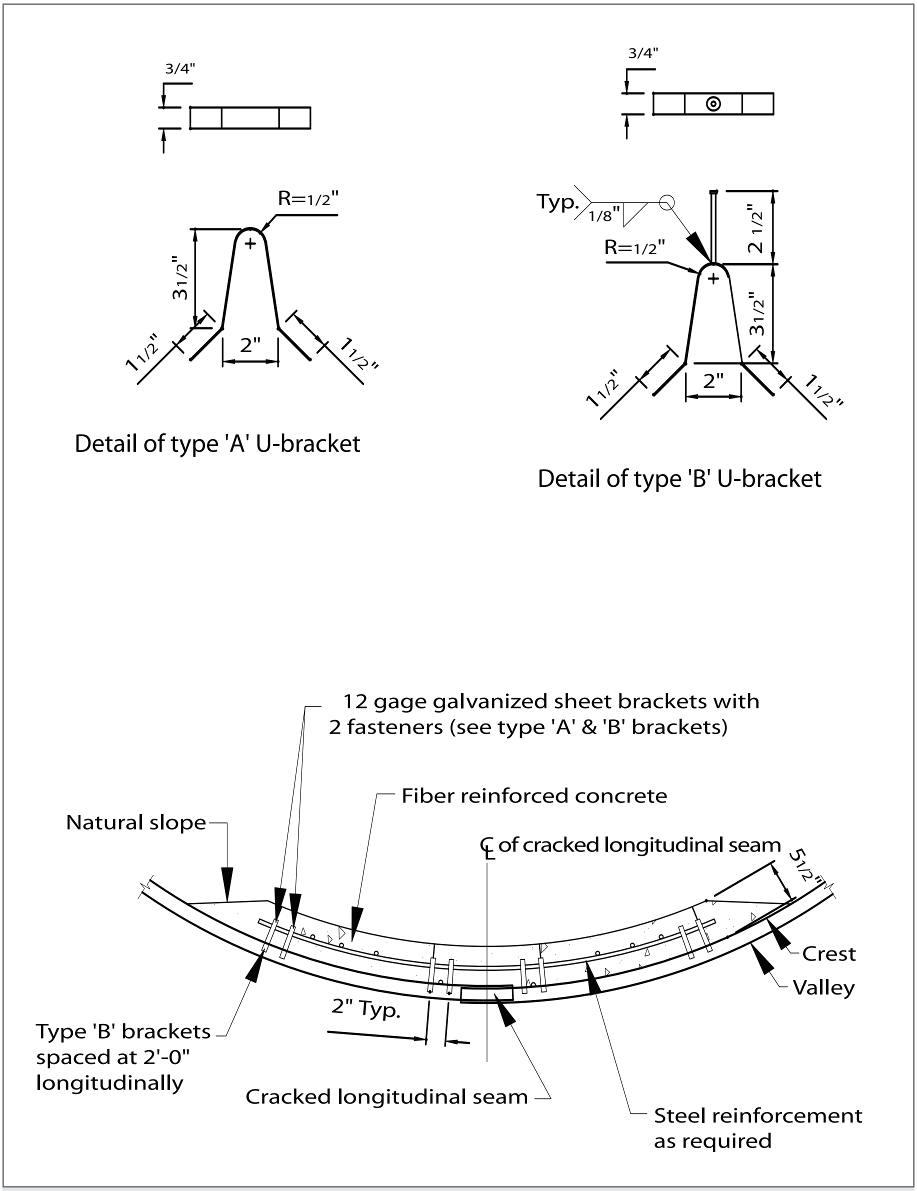


Figure 12.20 Partial shotcrete ring.

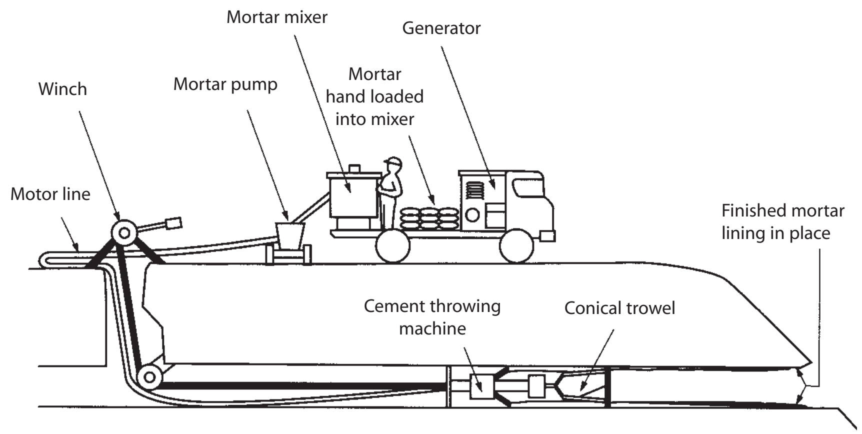


Figure 12.21 Cement mortar lining.

Other Repair and Rehabilitation Techniques and Considerations

Temporary Props

One of the most effective and expedient measures to ensure that excessive deformations of the pipe do not degenerate into sudden collapse is the provision of temporary struts or props in the pipe. These props can be timber columns of about 8 inches x 8 inches cross section, or steel struts of hollow circular section of the kind used in construction formwork. The props are located in the pipe under the crown and are provided with longitudinal sills above and under them. The sills, which run along the pipe length, are typically timber. When the sides of the pipe cross section are also excessively flattened, the vertical props are supplemented with horizontal supports.

The main advantage of vertical props is that they can prevent a catastrophic failure of the structure; the main disadvantage is that they constrict water flow. This disadvantage can be particularly significant for culverts. The props should be designed to carry, with an adequate margin of safety, the weight of that volume of soil apportioned to them. The props are usually spaced at 3 to 5 feet. The butt joints of the top and bottom sills should be staggered so that they do not occur at the same location along the pipe. The sills should be long enough to contain at least two props. Screws for adjusting the lengths of these props are very effective in ensuring that the contact between the supports and the pipe is not loose. Hydraulically, actions need to be taken to ensure debris does not collect on the structural system and clog the pipe.

Patching

Numerous patching compounds are commercially available. Compounds such as epoxies, which are used in bridge and paving repair can be used. Low shrink grout, plain and reinforced concrete, and polymerized asphalt are candidate materials that can be used to plug leaks, holes or restore inverts. A number of the above procedures are applicable to both concrete and steel pipe. However, use of welding and mechanical fasteners for repair is applicable only to steel pipe. Thus, the ease of maintenance associated with steel is a major factor in economical culvert and storm sewer design.

Partial Concreting Outside The Pipe

When distress in the pipe wall is limited to only the top segments of the pipe and the depth of cover is shallow, removal of the backfill from above the pipe and adding a layer of concrete to the outside of the pipe may prove to be an economically viable repair method. In this method, the concrete layer may be made composite with the pipe through the usual shear studs employed in slab-on-girder-type bridges. Where needed, shear studs can be machine-welded readily to the pipe after locally grinding off the zinc layer. The shear studs are staggered for maximum efficiency.

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Definition of Terms:

Many terms in this handbook are common to drainage, highway, and other related design and construction disciplines. Most of these are defined, described or illustrated where they appear in the book. However, to aid the engineering student and to clear up unfamiliar words for the professional engineer, a number of terms are here defined even though they may be elementary. For other unfamiliar terms, many are keyed in the index of this book, particularly where the definitions already appear in the text.

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A

abrasion—Wear by hydraulic traffic.

abutment—A wall supporting the end of a bridge or span, and sustaining the pressure of the abutting earth.

aerial sewer—An unburied sewer (generally sanitary type) supported on pedestals or bents to provide a suitable grade line.

aggradation—Progressive raising of the general level of a channel bed over a period of years by an accumulation of sediment.

allowable headwater elevation—The maximum permissible elevation of the headwater at a culvert at the design discharge.

allowable headwater depth—The depth corresponding to the allowable headwater elevation, measured from the invert at the first full cross section of the culvert.

allowable fish passage velocity— The maximum velocity fish can tolerate when passing upstream through a culvert.

anchor bolt—A foundation bolt; a drift spike, or any other device used for holding any mechanism or structure down. It may or may not be threaded.

angle—A rolled piece of steel having a cross section shaped into a right angle.

angle of repose—The angle which the sloping face of a bank of loose earth, or gravel, or other material makes with the horizontal.

anti-seep collar—see diaphragm

apron—Protective material laid on a stream bed to prevent scour at a culvert outlet, abutment, pier, toe of a slope, or similar location. (see also end section)

arch—Structural plate corrugated steel pipe formed to an arch shape and placed on abutments. The invert may be the natural stream bed or any other suitable material but is not integral with the steel arch.

arching—The effect produced by transfer of pressure between adjoining soil masses which settle relative to each other. Positive or active arching is that which results in the transfer of loads away from the conduit; negative or passive arching produces the opposite effect.

armor stone—A layer of stone protecting erodible material underlying the bed of a channel.

asphalt coating—Dipping corrugated steel pipe products, in a bath of hot asphalt for protection.

B

backfill—Earth or other material used to replace material removed during construction, such as in culvert, sewer and pipeline trenches and behind bridge abutments and retaining walls. Also refers to material placed in binwalls or between an old structure and a new lining.

backfill density—Percent compaction for pipe backfill (required or expected).

backwater—The rise of water level upstream due to an obstruction or construction in the channel.

backwater curve—Term applied to the calculation of the piezometric line from the obstruction.

baffle —A flow interference structure usually in the form of a low weir, which is attached to a culvert invert and extends partially or entirely across the culvert width. Baffle designs are constructed to aid in fish passage through the culvert barrel, or the channel.

band coupling—A collar or coupling which fits over adjacent ends of pipe to be joined, which when drawn tight, holds the pipe together by friction or by mechanical means. Types commonly available include: universal, corrugated. semi-corrugated, channel, flat, wing channel and internal expanding.

base (course)—A layer of specified or selected material of planned thickness, constructed on the sub-grade (natural foundation) or sub-base for the purpose of distributing load, providing drainage, or upon which a wearing surface or a drainage structure is placed.

batter—The slope of inclination from a vertical plane - as the face or back of a wall.

bedding—The earth or other material on which a pipe or conduit is supported.

bed load—Sediment in the flow that moves by rolling, sliding, or skipping along the bed; and is essentially in contact with the stream bed.

bend section—Intersection of the fall slope and barrel slope in a slope-tapered inlet.

bent protection system—Casing of structural plate or corrugated steel pipe installed to protect pile or framed bents.

berm—The space between the toe of a slope and excavation made for intercepting ditches or borrow pits.

- An approximately horizontal space introduced in a slope.
- Often used for word "shoulder" in road work.

beveled end—A cut-end treatment for structural plate products. The cut is on a plane inclined to the horizontal.

beveled inlet—A large chamfer or flare on the inlet edge of a culvert to improve the inlet coefficient Ke. Usually cast-in-place.

binwall—A series of connected bins, generally filled with earth or gravel to serve as a retaining wall, abutment, pier, or as protection against explosions or gunfire. (See Chapter 11)

bituminous (coating)—Of or containing bitumen; as asphalt or tar.

blue-green concept—The provision of stormwater detention ponds or lakes as an integral part of a park or greenbelt. In urban design, culvert sizing at roadways may be used to create temporary storage in the channel.

boring—An earth-drilling process used for installing conduits or pipelines.

box beam—Steel guardrail consisting of box sections cold formed from steel tubes.

box culvert—Drainage structure fabricated with standard structural plate reinforced with circumferential ribs having straight side legs bolted to corner plates curved to a small radius and a crown of large radius plates.

bridge plank (deck flooring)—A corrugated steel sub-floor on a bridge to support a wearing surface.

buoyancy—The power of supporting a floating body, including the tendency to float an empty pipe (by exterior hydraulic pressure).

burst speed—The swimming speed a fish can maintain for only a few seconds or for short distances without gross reduction of performance.

C

caisson—A watertight box or cylinder used in excavating for foundations or tunnel pits—to hold out water so concreting or other construction can be carried on.

camber—Rise or crown of the center of a bridge, or Bowline through a culvert above a straight line through its ends. A measure of adjustment required in the longitudinal profile of the bedding, in order to compensate for post-construction settlement. See Index.

cantilever—The part of a structure that extends beyond its support.

catch basin—A receptacle for diverting surface water to a sewer or subdrain, having at its base a sediment bowl to prevent the admission of grit and other coarse material into a sewer.

cathodic protection—Preventing corrosion of a pipeline by using special cathodes (and anodes) to circumvent corrosive damage by electric current.

- Also a function of zinc coatings on iron and steel drainage products—galvanic action.

channel treatment—Refers to the design to improve flow, or to reduce scour and/or erosion in the channel above or below the culvert. This, may include debris barriers before the inlet; paving or rip-rap to accelerate or decelerate flow velocity; training walls to direct flow; channel linings such as gabions, gobimats, special grasses, etc.; special inlet designs to improve or upgrade culvert capacity; special outlet designs for velocity scour prevention and/or energy dissipation; tailpond level control weirs for fish passage; and fish ladders above or below the culvert, or inside the culvert barrel.

chute—A steeply inclined channel for conveying water from a higher to a lower level.

closed invert culvert—A culvert having an invert which is structurally integral with the walls.

coefficient of runoff—Percentage of gross rainfall which appears as runoff. Also ratio of runoff to depth of rainfall.

cofferdam—A barrier built in the water so as to form an enclosure from which the water is pumped to permit free access to the area within.

cohesive soil—A soil that when unconfined has considerable strength when air-dried, and that has significant cohesion when submerged.

collar—An end treatment for a culvert, usually consisting of a concrete ring surrounding a cut-end treatment. The collar is usually attached to a cutoff wall.

combined sewer—A sewer that carries both storm water and sanitary or industrial wastes.

compaction—The densification of a soil by means of mechanical manipulation.

competent velocity—The velocity of water which can just move a specified type or size of material on a streambed.

conduit—A pipe or other opening, buried or above ground, for conveying hydraulic traffic, pipelines, cables or other utilities.

consolidation—The gradual reduction in a volume of a soil mass resulting from an increase in compressive stress.

conventional culvert—A closed invert culvert having no major inlet improvements such as a side-tapered or slope-tapered inlet. It may incorporate minor improvements such as, cut-end treatments, beveled edges, wingwalls, a fall, or a prefabricated end section.

conveyor conduit—Corrugated steel structures of varying diameters used to enclose a conveyor system.

conveyor cover—Half circle steel arch sections supported on bandsheets which are bolted to the conveyor frame.

conveyor tunnel—Usually a large diameter structural plate pipe installed to enclose a materials handling system. Commonly used under aggregate piles.

cooling water intake or discharge lines—A large diameter conduit carrying cooling water to a power plant and heated return water to the source. These lines are usually subaqueous requiring special underwater installation by divers.

corrugated steel pipe (CSP)—Galvanized sheet steel formed to finished shape by the fabricator:

- **riveted**—A corrugated steel pipe with annular corrugations, fabricated from cut-to-length corrugated steel pipe sheet with lapped longitudinal and circumferential seams fastened with rivets.
- **double wall**—A full circular cross section pipe helically formed with an outer corrugated shell and integrally seam-connected with an inner liner of smooth or uncorrugated steel sheet.
- **helical**—Corrugated steel pipe with helical corrugations, fabricated from coiled corrugated steel pipe sheet, with a continuous helical seam, either lock or welded.
- **spiral rib**—A full circular cross-section pipe with a single thickness of smooth sheet, fabricated with helical ribs projecting outwardly.

corrugated steel pipe sheet—A mill product in sheet or coil form for fabricating riveted or helical corrugated steel pipe products, galvanized by the continuous hot-dip process. (abbrev. "CSP sheet")

cost effective—Answering the purpose of providing the optimum effect at the most reasonable cost.

coupler—See band coupling.

critical density—Zone separating the levels of backfill compaction that will and will not prevent deflection failure of a pipe (Between 70% and 85% standard density).

critical depth—Depth of flow at which specific energy is a minimum for a given flow. Water depth in a conduit at which certain conditions of maximum flow will occur. Other conditions are: (1) the conduit is on a critical slope with water flowing at critical velocity, (2) there is an adequate supply of water.

critical flow—A condition that exists at the critical depth, and where the sum of the velocity head and static head is a minimum. Also, that flow which has a Froude number of one.

critical migration delay period—The maximum delay fish can tolerate during the spawning migration without harmful biological consequences.

critical slope—The slope at which a maximum flow will occur at minimum velocity. The slope or grade that is exactly equal to the loss of head per foot resulting from flow at a depth that will give uniform flow at critical depth.

critical velocity—Mean velocity of flow when flow is at critical depth.

crown—The highest point on a transverse section of conduit. (see soffit) Also the highest point of a roadway cross section.

culvert—A culvert is a conduit for conveying surface water through an embankment. It is a "grade separation" for water and the traffic or facility above it. The embankment may be for a highway, railway, street, industrial roadway, spoil bank, dam or levee.—A distinction is made between culverts and storm sewers, mostly on the basis of length and the types of inlets and outlets. Distinction is also made between culverts and bridges in that the top of a culvert does not serve as a road surface, whereas a bridge is a definite link in a roadway surface. Culverts larger than about 5 to 8 meter span are usually referred to as "soil-steel bridges", to connate the need for greater engineering involvement.

culvert uplift—The upward movement of a culvert end, resulting from hydraulic and buoyancy forces.

cut-end treatment—Refers solely to what is done to the steel inlet or outlet. May be standard pipe bevels, or pipe arch bevels, or skew cuts. Combinations of bevels and skews are not recommended practice. (See end treatment and slope treatment.)

cutoff wall—A wall intended to prevent seepage or undermining (see diaphragm). Usually a buried vertical wall below the end of a culvert.

D

dBA—See weighted decibel.

deadman—Buried anchorage for a guy, cable, etc. Commonly used in retaining walls, cutoff walls, piling and other designs.

debris—Any material including floating woody materials, and other trash, suspended and moved by a flowing stream.

degradation—The progressive general lowering of a stream channel by erosion, other than that caused by a constriction.

depression storage—The fraction of precipitation that is trapped in depressions on the surface of the ground, with the only outlet through infiltration or evaporation.

depth-of-cover—The vertical distance between the profile grade and the crown. Serves as basis for calculation of dead load on structure.

depth-of-scour—The depth of material removed from a stream bed by scour, measured below the original bed elevation.

design discharge—A quantity of flow that is expected at a certain point as a result of a design storm, or flood frequency. Usually expressed as a rate of flow in cubic feet per second, or cubic meters per second. Also the discharge which a structure is designed to accommodate without exceeding the adopted design constraints.

design frequency—The recurrence interval for hydrologic events used for design purposes. As an example, a design frequency of 50 years means a storm of a magnitude that would be expected to recur on the average of once in every 50 years.

design life—The length of time for which it is economically sound to require a structure to serve without major repairs, or replacement.

design storm—A precipitation event that, statistically, has a specified probability of occurring in any given year (expressed either in years or as a percentage). May also be a particular storm that contributes runoff for which the drainage facilities were designed to handle.

detention storage—Temporary storage of excess runoff in surface ponds, or underground tanks, for the purpose of attenuating excess runoff.

detritus—Rock, gravel, sand, silt or other materials carried by flowing water.

diameter—Inside diameter, measured between inside crests of corrugations.

diaphragm—A metal collar at right angles to a drain pipe for the purpose of retarding seepage or the burrowing of rodents. Often specified on pipe spillways, or other drainage structures designed to operate under static head, or head ponding at the inlet.

dike—An embankment or wall constructed to prevent flooding.

discharge—The actual volume of water flowing from a drainage structure per unit of time. Usually measured in cubic feet per second or cubic meters per second.

ditch check—Barrier placed in a ditch to decrease the slope of the Bowline and thereby decrease the velocity of the water.

drainage—Interception and removal of ground water or surface water, by artificial or natural means.

drop structure—A structure in a channel or conduit which permits water to drop to a lower level.

dry well—A steel catch basin with open bottom and perforated walls, that is used to store surface runoff for infiltration, or recharge, into the ground.

E

EOS—Equivalent Opening Size, a major parameter in the selection of a filter fabric for use in filtration and separation.

effluent—Outflow or discharge from a sewer or sewage treatment equipment.

ellipsed—With reference to structural plate corrugated steel pipe, factory-formed to an elliptical shape. May be vertical or horizontal ellipse. Term "elongated" usually refers to a 5% vertical ellipse shape.

embankment (or fill)—A bank of earth, rock or other material constructed above the natural ground surface.

embedment—The depth to which a culvert invert is embedded below the natural stream bed.

end area—The area calculated on the basis of inside diameter (See diameter); or the available flow area through the conduit.

end section—Flared metal attachment on inlet and outlet of a culvert to prevent erosion of the roadbed, improve hydraulic efficiency, and improve appearance. See Index.

end treatment—The overall design of a culvert inlet and/or outlet. This may involve channel treatments, cut end treatments, slope treatments, headwalls, anchorage, etc.

energy dissipator—A structure used to dissipate the energy possessed by high-velocity flow at the outlet of a culvert.

energy grade line—A hydraulic term used to define a line representing the total amount of energy available at any point along a water course, pipe, or drainage structure. Where the water is motionless, the water surface would coincide with the energy grade line. As the flow of water is accelerated the water surface drops further away from the energy grade line. If the flow is stopped at any point the water surface returns to the energy grade line. The energy grade line is established by adding together the potential energy as the water surface elevation (referenced to a datum); and the kinetic energy (usually expressed as a velocity head), at points along the channel or conduit profile.

energy gradient—Slope of a line joining the elevations of the energy head of a stream.

energy head—The elevation of the hydraulic gradient at any section, plus the velocity head.

engineered soil—A selected soil of known properties placed around a conduit in a prescribed manner.

entrance head—The hydraulic head required to cause flow into a conduit; it includes both entrance loss and velocity head.

entrance loss—The head lost in eddies and friction at the conduit inlet.

equalizer—A culvert placed where there is no channel but where it is desirable to have standing water at equal elevations on both sides of a fill.

equivalent diameter—The diameter of a round corrugated steel pipe from which a pipe arch or other shape is formed.

erosion—Wear or scouring caused by hydraulic traffic or by wind.

F

fabricator—A manufacturer of corrugated steel pipe or structural plate corrugated steel pipe product, or other steel construction products. Premises of a manufacturer are referred to as the fabricating plant.

face section—The upstream face of the enlarged and fully enclosed opening of an improved inlet.

fall—A steeply inclined length of channel in or immediately upstream from a culvert inlet to improve hydraulic capacity.

fan duct—Mine ventilation system in which a conduit extends from the ventilating fan to the portal of the fresh air tunnel or air shaft.

fiber-bonded protected corrugated steel pipe—A mill product in which an aramid nonwoven fabric is embedded in the zinc coating, followed by asphalt coating.

filter—Granular material placed around a subdrain pipe to facilitate drainage and at the same time strain or prevent the admission of silt or sediment.

filter cloth—See geotextiles.

fishway—A facility to permit fish to pass an obstruction with minimum stress.

flap gate—A hinged watertight flap covering the outlet of a culvert to allow outflow from the culvert but prevent backflow resulting from higher flood stages downstream.

flexibility factor (FF)—Relative elastic deflection of a conduit. See Chapter. 4, Structural Design. Equation 9.

flood—A relatively high flow, in terms of either water level, or discharge.

flood plain—The relatively level land which adjoins a water course, and which is subject to periodic flooding, unless protected artificially by a dike, or similar structure.

flood routing—An analytical technique used to compute the effects of system storage (i.e. detention ponds); and system dynamics on the timing and shape of a flood wave at recessive points along a stream or channel.

flow area—See end area.

flume—An open channel or conduit of metal, concrete or wood, on a prepared grade, trestle or bridge.

ford—A shallow place where a stream may be crossed by traffic.

foundation—That portion of a structure (usually below the surface of the ground) which distributes the pressure to the soil or to artificial supports. Footing has similar meaning.

foundation drain—A perforated CSP, or a system of CSP subdrains which collects ground water from the foundation or footings or engineered structures, for the purpose of draining unwanted waters away from such structures.

freeboard—The height from a design water level to the top of an embankment, roadway, dam or wall.

free field overburden pressure— The vertical earth pressure at a level in a semi-infinite mass, due to the load of earth and other materials above that level.

free outlet—(as pertaining to critical flow)—Exists when the backwater does not diminish the discharge of a conduit.

free water—Water (in soil) free to move by gravity (in contrast to capillary or hydroscopic moisture).

G

gabion—A steel wire mesh basket filled with stones or broken concrete, and forming part of a larger unit of several such baskets, usually for channel or end treatment, for erosion or scour control, or other purposes.

gage—Reference system for thickness of metal sheets or wire (and bearing a relation to the weight of the metal).

gaskets—A thin sheet of rubber, sheet metal, or other materiel forming a joint between two pieces of metal to prevent leakage. Gaskets for corrugated steel pipe are O-ring, sleeve, or strip type.

geotextiles—Woven or nonwoven engineering fabrics that act as separators to keep soil or fines out of a subdrainage piping system while serving as a filter to allow free flow of water.

gradation—Sieve analysis of aggregates.

grade—Profile of the center of a roadway, or the invert of a culvert or sewer. Also refers to slope, or ratio of rise or fall of the grade line to its length. (Various other meanings)

grade separation—A corrugated steel structure, usually structural plate, installed to allow passage of a road or railroad over another road or railroad. An underpass.

gradient (slope)—The rate of rise or fall of a grade—expressed as a percentage or ratio as determined by a change in elevation to the length.

granular—Technical term generally describing the uniformity of grain size of gravel, sand or crushed stone.

groin—A jetty built at an angle to the shore line, to control the waterflow and currents, or to protect a harbor or beach.

ground water table (level)—Upper surface of the zone of saturation in permeable rock or soil. (When the upper surface is confined by impermeable rock, the water table is absent.)

grout—A fluid mixture of cement, sand, and water that can be poured or pumped easily.

guiderail—A barrier located along the edge of a roadway shoulder for the purpose of guiding errant vehicles onto the roadway.

H

haunch—The portion of the conduit cross-section between the spring line and the top of the bedding or footing.

head (static)—The height of water above any plane or point of reference. (The energy possessed by each unit of weight of a liquid, expressed as the vertical height through which a unit of weight would have to fall to release the average energy possessed.) See Chapter. 3, Hydraulics. Standard unit of measure shall be the foot. Relation between pressure expressed in psi and feet of head is:

$$\text{Head in feet} = \frac{\text{psi x 144}}{\text{Density in}^{\text{lb/cu ft}}}$$

for water at 68° F
1 psi = 2.310 ft.

headwall—A wall (of any material) at the end of a culvert or drain to serve one or more of the following purposes: to protect fill from scour or undermining, increase hydraulic efficiency, divert direction of flow, and/or serve as a retaining wall. Usually a separate vertical cutoff wall at the inlet, or outlet. May be square end, or wing wall, or cribwall design of varying heights; and in a steel, concrete, or masonry. Usually constructed or installed before or during backfill. A partial headwall is less than the full rise of the culvert. (See also end treatment, slope treatment, cutoff wall, cut-end treatment and improved inlet.)

headwater elevation—The water level upstream from a structure.

heat manifold—A corrugated steel pipe installed in an aggregate pile to allow passage of heat to help obtain satisfactory working and setting properties in concrete.

height of cover (HC)—Distance from crown of a culvert or conduit to (1) for highways, bottom of flexible pavement or top of rigid pavement (2) for railways, bottom of tie.

high profile arch—A corrugated steel structure with a relatively high rise in relation to span.

hook bolt—A bolt having one end in the form of a hook.

horizontal ellipse—A long span corrugated steel structure with the major diameter horizontal.

hydraulic gradient—A line which represents the relative force available due to the potential energy available. This is a combination of energy due to the height of the water and internal pressure. In an open channel, the line corresponds to the water surface. In a closed conduit, if several openings are placed along the top of the pipe and open end tubes inserted, a line connecting the water levels in the tubes represents the hydraulic grade line.

hydraulic jump—Transition of flow from the rapid to the tranquil state. A varied flow phenomenon that produces a rise in elevation of backwater flow surface. A sudden transition from supercritical flow to subcritical flow, conserving momentum and dissipating energy.

hydraulic radius—The cross-sectional area of a stream of water divided by the length of that part of its periphery in contact with its containing conduit; the ratio of area to wetted perimeter.

hydraulics—That branch of science or engineering which treats the mechanical properties of water or other fluid motion.

hydrogen ion (pH)—Refers to acidity or alkalinity of water or soil. An ion is a charged atom or group of atoms in solution or in a gas. Solutions contain equivalent numbers of positive and negative ions.

hydrograph—A graph of runoff rate, inflow rate, or discharge rate, versus time.

hyteograph—A graph showing average rainfall, rainfall intensities, or rainfall volume over specified areas, with respect to time.

I

ice jam—The choking of a stream channel by the piling up of drift ice at an obstruction or water course constriction.
icing—The gradual accumulation of ice in a channel or culvert, resulting from freezing of ground water seepage over a period of weeks or months.

impervious—Impenetrable. Completely resisting entrance of liquids.

improved inlet—A culvert inlet incorporating geometry refinements, other than those used in conventional culvert practice, for the purpose of improving the culvert capacity. (see headwall)

infiltration—The passage of water into the soil. The term is also used to refer to groundwater entering a sewer system through joints, manholes, etc. infiltration is not usually desirable in sanitary sewer systems, but may be desirable in urban storm drain systems to control groundwater table, and protect roadway pavements.

inflow—The water discharged into a sewer system from all possible sources, but not infiltration.

inlet control—A hydraulic term which indicates that the capacity of the conduit is governed by the quantity of water which the inlet will accept, due to its size and geometry, and the nature and depth of the head pond. Flow control at a culvert in which the capacity is governed by the inlet characteristics and head-water depth only.

inlet time—The time required for runoff to flow from the most remote point of a drainage area to a point where it enters the sewer.

interaction (soil-steel)—The division of load carrying between pipe and backfill and the relationship of one to the other.

intercepting drain—A ditch or trench filled with a pervious filter material around a subdrainage pipe.

invert—That part of a pipe or sewer below the spring line—generally the lowest point of the internal cross section. The stream bed or floor within a structure or channel.

invert paving—The bottom portion of a pipe conduit that is paved with a material to improve flow, erosion and corrosion characteristics. Asphalt is commonly used for CSP products, and wire mesh and concrete for larger structural plate structures.

inverted pear—A long span structure in which the rise is the major dimension.

J

jacking (for conduits)—A method of providing an opening for drainage
or other purposes underground, by cutting an opening ahead of the pipe and forcing the pipe into the opening by means of horizontal jacks.

L

lateral—A conduit diverting water from a main conduit, for delivery to distributaries; a secondary ditch.

lift—One layer of soil material placed in the backfilling process.

liner plate—Formed steel unit used to line or reinforce a tunnel or other opening.

lock seam—Helical seam in a pipe, formed by overlapping or folding the adjacent edges.

low profile arch—A long span structure in which the span is the major dimension.

luminaire—In highway lighting, a complete lighting device consisting of a light source, plus a globe, reflector, refractor, housing. and such support as is integral with the housing. The light standard (bracket or pole) is not considered a part of the luminaire.

M

major system—The route followed by storm runoff when the minor system is either inoperative or inadequate. It generally consists of roads, swales, and major drainage channels. The major system is generally designed to provide 25 to 100 yr. protection against surface flooding.

manhole—Opening from street surface to provide entry for inspection and cleaning of sewer lines.

Manning's Formula—An equation for the value of coefficient C in the Chezy Formula, the factors of which are the hydraulic radius and a coefficient of roughness.

mean velocity—Average velocity within a stream or conduit cross section.

median—The portion of a divided highway separating roadways.

median barrier—A double-faced guiderail in the median or island dividing two adjacent roadways.

metallic coating—A zinc or aluminum coating applied to corrugated steel pipe for corrosion protection.

minor system—The traditional storm runoff design of storm sewers, street gutters, roof leader connections, foundation drains, etc.—designed to convey runoff from frequent, less intense storms, to eliminate or minimize inconvenience in the area to be developed. (See major system.)

miter cut—An angle in the barrel. A wedge section is cut from the barrel, and the barrel welded to provide a change in alignment. Permits pipe curvature, or changes in grade and/or alignment.

mitered end—A culvert end the face of which conforms roughly with the face of the embankment slope. (see also the preferred term "beveled end")

N

nestable pipe—Half round corrugated steel pipe segments joined by interlocking notches or mating flanges. Primarily used for encasing existing utility or other lines.

noise barriers—All-steel sound reflective barrier located between the source of noise and the desired quiet zone.

nominal thickness—The order thickness for the steel sheet or plate.

normal design flood—The design flood used for the hydraulic design of structures, in the absence of imposed criteria, such as the regulatory flood.

normal water level—The average summer water level. The free surface associated with flow in natural streams.

O

open channel—A drainage course which has no restrictive top. It is open to the atmosphere and may or may not permit surface flow to pass over its edge and into another channel in an unrestricted manner. In many cases where dikes or beams are constructed to increase channel capacity, entrance of surface waters is necessarily controlled.

outfall (outlet)—In hydraulics, the discharge end of drains and sewers.

outlet control—Flow control at a culvert in which the capacity is governed principally by the barrel roughness, length and slope, and in some cases by the tailwater.

P

parapet—Wall or rampart, chest high. Also, the wall on top of an abutment extending from the bridge seat to the underside of the bridge floor and designed to hold the backfill.

paved invert—A smooth asphalt pavement that completely fills the corrugations of the lower segment of a pipe; intended to provide resistance to erosion, and to improve flow.

pear—See inverted pear.

perforated pipe—A corrugated flood used for the steel pipe product with perforations completely through the pipe walls.

> **fully perforated**—A pipe with perforations around the periphery, usually for recharge to ground of storm water.
>
> **invert-perforated**—A pipe with perforations in the lower segment, usually for subdrainage.

performance curve—A plot of discharge versus headwater elevation or depth at a culvert.

periphery—Circumference or perimeter of a circle, ellipse, pipe arch, or other closed curvilinear figure.

permeability—A property of soils which permits free passage of any fluid. Permeability depends on grain size, void ratio, shape and arrangement of pores. Often referred to as penetrability.

pervious—Applied to material through which water passes relatively freely. (i.e. sands and gravels)

pile, bearing—A member driven or jetted into the ground and deriving its support from bearing on the underlying strata and/or by the friction of the ground on its surface. (See also Sheeting)

pipe—A culvert having a non-rectangular cross-section, often assumed to be circular unless specified otherwise.

pipe arch—A corrugated steel pipe or structural plate corrugated steel pipe shaped to a span greater than rise; a multi-radius shape with an arch-shaped top and a slightly convex integral bottom, structurally continuous with an invert whose radius of curvature is greater than that of the crown.

piping—Subsurface erosion caused by the movement or percolation of water fill, or natural ground.

plate—A flat-rolled steel product. See structural plate.

polymeric coating—A plastic coating, bonded to one or both sides of the CSP sheet, prior to fabricating into pipe.

pending—The use of water to hasten the settlement of an embankment—requires the judgment of a soils engineer. In hydraulics, ponding refers to water backed up in a channel or ditch as the result of a culvert of inadequate capacity or design to permit the water to flow unrestricted.

precipitation—Process by which water in liquid or solid state (rain, sleet, snow) is discharged out of the atmosphere upon a land or water surface.

profile grade—The top of the finished granular base of the centerline of the highway or railway.

projecting end—A culvert end which protects from the face of the embankment.

protective coating—A coating applied to the pipe in addition to the standard zinc protection, such as asphalt, polymeric, and aramid fibers.

R

rational method—An empirical approach to estimate storm runoff, by use of the formula Q = CIA, where C is coefficient describing the runoff potential of a drainage area. I is the rainfall intensity during the core time of concentration and A is the drainage area.

re-entrant arch—An arch with haunches.

reformed end—Annular corrugations rolled onto the ends of helically corrugated steel pipe.

regulatory flood—A flood designated for a specific site by a regulatory jurisdiction. or agency, generally for flood plain management purposes.

relief flow—A portion of a major flood which bypasses the main structure at a stream crossing, by flowing over the roadway, or through a relief bridge or culvert.

retaining wall—A wall for sustaining the pressure of earth or filling deposited behind it.

return period—The average period in years between occurrences of a discharge equalling or exceeding a given value.

revetment—A wall or a facing of wood, willow mattresses, steel units, stone, or concrete placed on stream banks to prevent erosion.

Reynolds' Number—A non-dimensional coefficient used as a measure of the dynamic scale of a flow.

ring compression—The principal stress in a confined thin circular ring subjected to external pressure.

riprap—Rough stone of various sizes placed compactly or irregularly to prevent scour by water or debris.

rise—The maximum vertical clearance inside a conduit at a given transverse section, usually the centerline.

roadway (highway)—That portion of the highway including the shoulders, for vehicular use. A divided highway has two or more roadways. (railway)—That part of the right of way prepared to receive the track. (During construction the roadway is often referred to as the "grade.")

rodent gate—An appurtenance at the outlet end of a subdrain or other drainage pipe that swings outward to permit flow and detritus to pass, yet prevents the passage into the drainage network of rodents or other animals, whose nesting could block, and render inoperative, the drain system.

roof leader—A drain or pipe that conducts storm water from the roof of a structure downward and into a sewer for removal from the property, or onto or into the ground for seepage disposal.

roughness coefficient (n)—A factor in the Kutter, Manning, and other flow formulas representing the effect of channel (or conduit) roughness upon energy losses in the flowing water.

runoff—That part of precipitation carried off from the area upon which it falls. AIM the rate of surface discharge of the above. That part of precipitation reaching a stream, drain or sewer. Ratio of runoff to precipitation is a "coefficient" expressed decimally.

S

scour—The local lowering of a stream bed by the erosive action of flowing water.

> **general scour**—is that which occurs in a waterway opening as a result of obstruction of the flow.
>
> **local scour**—is that which occurs at a pier or abutment as a result of local obstruction to the flow.
>
> **natural scout**—is the scour of a stream bed resulting from natural phenomena, such as channel meandering.

seam—A joint between two structural steel plates formed by overlapping and bolting them together. Also the join or lap of riveted CSP. Also the join or weld for continuous helical-weld CSP. (see also lock seam)

sediment—Soils or other materials transported by wind or water as a result of erosion.

seepage—Water escaping through or emerging from the ground along some rather extensive line or surface, as contrasted with a spring, the water of which emerges from a single spot.

service tunnel—A conduit connecting two buildings to provide more direct access for employees, products, materials, or utility lines.

shaft—A pit or well sunk from the ground surface into a tunnel for the purpose of furnishing ventilation or access to the tunnel.

sheathing—A wall of metal plates or wood planking constructed to maintain trench wall stability.

sheeting—A wall of metal plates or wood planking to keep out water, or soft or runny materials.

shoulder—The portion of the conduit between the crown and the spring line.

side tapered inlet—An 'improved' inlet having an enlarged face area with the transition to the culvert barrel accomplished by tapering the sidewalls. Both the barrel and the enclosed inlet structure are on the same grade. Usually cast-in-place.

sill—A low wall placed transversely in a culvert or channel level with or slightly above the invert. Often used downstream of the culvert to maintain tailpond level.

sheet flow—Water flowing across a wide, flat paved area such as a highway or parking lot; may result from rainfall or melting ice or snow.

siphon (inverted)—A conduit or culvert with a U or V shaped grade line to permit it to pass under an intersecting roadway, stream or other obstruction.

skew (skew angle)—The acute angle formed by the intersection of the line normal to the centerline of the road improvement with the centerline of a culvert or other structure.

skew number—The angle between the highway centerline and the culvert centerline measured clockwise, and specified in increments of 5.

slide—Movement of a part of the earth under force of gravity.

slope-tapered inlet—A cast-in-place side-tapered inlet, incorporating a fall within the enclosed inlet structure.

slope treatment—Describes what is done to protect the embankment slope from scour or erosion. May be vegetation (i.e. Bermuda grass); grouted masonry or rip-rap; a "donut" type concrete collar with entrance flare to improve the inlet coefficient, usually from the headwall up over the crown (and usually on bevel ends, with embedded hookbolts in the casting); plus others. Always placed or constructed after backfill.

slotted steel pipe—Corrugated steel pipe with reinforced longitudinal slots at the crown. Used for interception of sheet flow. The system provides an inlet, runoff pipe and grate in a single unit. Pipe can be perforated for use as an underdrain.

smooth-lined asphalt—A smooth asphalt interior lining that completely fills the corrugations in an asphalt coated corrugated steel pipe. (see also spun lining)

soffit—The bottom of the top of a pipe. In a sewer pipe, the uppermost point on the inside of the structure. The crown is the uppermost point on the outside of the pipe wall.

soil liquefaction—Loss of strength of a soil resulting from the combined effects of vibrations and hydraulic forces, thereby causing the material to flow.

soil-steel structure—A bridge, comprised of structural steel plates and engineered soil, designed and constructed to induce a beneficial interaction of the two materials.

span—Horizontal distance between supports, or maximum inside distance between the sidewalls.

speller—Zinc or galvanized coating on steel products.

spillway—A low-level passage serving a dam or reservoir through which surplus water may be discharged; usually an open ditch around the end of a dam, or a gateway or a pipe in a dam.

- An outlet pipe, flume, or channel serving to discharge water from a ditch, ditch check, gutter or embankment protector.

spread footing—A footing which transfers load directly to the underlying foundation material. Used in structural plate arches.

spring line—The line of the outermost points on the side of a conduit. On a circular pipe, it would be the point one-half of the diameter above the invert. Also, the line of intersection between the intrados and the supports of an arch. Also, the maximum horizontal dimension of a culvert or conduit.

spun lining—An asphalt lining in a pipe, made smooth or uniform by spinning the pipe around its axis.

stable stream grade—The slope of a natural channel at which neither aggradation nor degradation occurs.

Standard Proctor Density—The optimum unit weight of a soil determined in accordance with ASTM designation D-698, or AASHTO T-99.

steady flow—A flow in which the volume passing a given point per unit of time remains constant.

storage basin—Space for detention or retention of storm runoff water for controlled release during or following design storm. Storage may be upstream, downstream, offstream, onstream and/or underground.

storage bin—Built from heavy, curved corrugated steel plates. Used on construction sites and plant storage sides for coal, sand, gravel and other materials.

storm sewer—A sewer that carries only storm water, or clear water runoff.

stormwater management—A master plan, or systems approach to the planning of facilities, programs, and management organizations for comprehensive control and use of stormwater within a defined geographical area.

stream check—A barrier placed in a stream to decrease the slope of the Bowline and thereby the velocity of the water. It is provided with a throat or spillway for dropping the water to a lower level.

stream enclosure—A pipe or other conduit for carrying a stream underground paralleling a roadway or dividing otherwise useful land into smaller parts.

structural plate corrugated steel pipe—Hot-rolled sheets or plate, corrugated, custom hot-dipped galvanized, curved to radius, assembled, and bolted together to form pipes, pipe arches, and other shapes.

subcritical flow—Flow at velocities less than critical, or with a Froude number less than one. In this state, the role played by gravity forces is more pronounced. so the flow has a low velocity, and is often described as steady, tranquil, or streaming.

sub drain—A previous backfilled trench containing a pipe with perforations or open joints for the purpose of intercepting ground water or seepage.

subdrainage—The control of groundwater. Subdrainage helps maintain stable subgrades and structure foundations, eliminates wet cuts and prevents frost heave.

subgrade—The surface of a portion of the roadbed on which paving, or railroad track ballast, or other structure is placed.

supercritical flow—Flow with a Froude number greater than one. In this state, the inertia forces become dominant, so the flow has a high velocity, and is usually described as rapid or shooting.

surcharge—The flow condition occurring in closed conduits when the hydraulic grade line is above the crown of the sewer.

T

tailwater—The water just downstream from a structure.

tailwater depth—The depth of water immediately downstream from a culvert, measured from the invert of the culvert outlet.

threading—The process of installing a slightly smaller pipe or arch within failing drainage structure.

throat section—The intersection of cast-in-place sidewall tapers and culvert barrel in a side—or slope-tapered 'improved' inlet.

thrust—The circumferential compressive force in the conduit walls, per unit length of conduit.

time of concentration—The time required for storm runoff to flow from the most remote point, of a drainage area to the point under consideration. It is usually associated with design storm.

toe drain—A subdrain installed near the downstream toe of a dam or levee to intercept seepage.

transverse section—A section in the vertical plane normal to the horizontal projection of the longitudinal direction.

trash rack—A pervious barrier constructed to catch debris, and prevent blockage of the inlet of a drainage conduit.

trunk (trunk line)—In a roadway or urban drainage system, the main conduit for transporting the storm waters. This main line is generally quite deep in the ground so that laterals coming from fairly long distances can drain by gravity into the trunk line.

tunnel lining—Added inner surface of a tunnel; can be concrete, brick, or steel. A bolted metal shell serving either as a permanent inner surface for a tunnel or as a form by which a concrete wall coating is built.

U

underdrain—See sub-drain.

undermining—A process of scour by hydraulic action that progressively removes earth support from an engineered structure. Undermining is commonly found at the outlet of a culvert or sewer.

underpass—An opening under a roadway to allow pedestrians, livestock, or other traffic to pass in safety. Also an opening under a railroad or other roadway through which a street, highway, or railroad passes.

uniform flow—Flow in which the velocities are the same in both magnitude and direction from point to point along the stream or conduit, all stream lines being parallel.

unsteady flow—A flow in which the velocity changes with respect to both space and time.

utilidor—Utility corridor. See utility conduits.

utility conduits—Conduit installed for the protection of water, steam and gas lines, sewers, or power cables passing underneath a building, roadway, or other obstacle.

V

value analysis—Objective analysis of the features and benefits of corrugated steel pipe in relation to a specified alternate.

velocity head (symbol H_v)—For water moving at a given velocity, the equivalent head through which it would have to fall by gravity to acquire the same velocity.

vertically ellipsed pipe—An elliptical conduit with major diameter vertical and not less than 1.10 times the minor diameter.

ventilation ducts—A conduit installed to provide various degrees of ventilation to protect against health hazards arising from nontoxic gasses, heat, dust, or moisture.

void forms—A corrugated steel tube installed in the concrete deck of a bridge to reduce the amount of concrete used and the overall weight of the deck.

W

wale—Guide or brace of steel or timber, used in trenches and other construction.

washout—The failure of a culvert bridge, embankment or other structure resulting from the action of flowing water.

water course—A natural or artificial channel in which a flow of water occurs, either continuously or intermittently. Natural water courses may be either on the surface, or underground.
water table—The upper limit of the portion of ground wholly saturated with water.

watershed— Region or area contributing to the supply of a stream or lake; drainage area, drainage basin, catchment area.

weighted decibel (dBA)—The most commonly used environmental noise unit. The A indicates that a frequency weighing has been applied within the sound measuring instrument to approximate the sensitivity of the human ear.

weir crest—The point of intersection of the upstream channel slope and the fall slope.

wetted perimeter—The length of the wetted contact between the water prism and the containing conduit, (measured along a place at right angles to the conduit).

Z

zero runoff increase—A concept in which the peak rate of storm runoff from a new urban development is limited to that which occurred prior to development.

zinc coating—A galvanic, barrier coating applied to the surfaces of steel sheet, plate, or other components.

SYMBOLS

Various disciplines of engineering, hydraulics, physics, chemistry, etc. have established standard symbols or letters to denote various factors or dimensions in formulas, tables, drawing and texts. Some of these are found in dictionaries; others have been published by technical associations. Some of the symbols used in this handbook are listed here. For others, reference should be made to sources such as are listed for the preceding Glossary.

Symbol	Definition or Use
a	Area, cross-sectional, culvert,
a	Constant in an Intensity-Duration Frequency Curve,
A	Area, cross-sectional, of waterway, ft^2,
A	Area of long span structure, ft^2,
A	Drainage area, acres,
A	Area of section, $in.^2$,
A	Width of roadway surface or roadbed in determining culvert length,
A	Required wall area,
A	Cross-sectional area of flow in ft^2 at right angles to the direction of flow,
A	Area to be subdrained, acres,
A	Cross-sectional area of liner plate, $in.^2/ft$,
A_c	Partial flow area,
b	Constant in an Intensity-Duration Frequency Curve,
b	Bottom width of a trapezoidal channel,
b	Developed width factor,
B	Invert to spring line,
B	Slope width from roadway to top of culvert on a flat grade,
B	Long span structure length, ft.
B_1	Slope width from roadway to top of upstream end of culvert on a steep grade,
B_2	Slope width from roadway to top of downstream end of culvert on a steep grade,
c	Constant in an Intensity-Duration Frequency Curve,
c	Coefficient of roughness whose value depends on the surface over which water flows,
C	Coefficient, runoff,
C	Compression in pipe wall,
℄	Centerline,
C	Long span dimension between centers of inside radii,
C	Ring compression, thrust, Ib/ft,
C	Elevation from bottom of culvert to top of roadway,
C	Subsurface runoff factor, ft^3/sec,
C_a	Recommended antecedent precipitation factor for the rational formula,

Symbol	Definition or Use
C_d	Soil coefficient for tunnel liner,
C_1	Difference in elevation from roadway surface to top of the upstream end of a culvert on a steep grade,
C_2	Difference in elevation from roadway surface to the top of the downstream end of a culvert on a steep grade,
CO	Carryover design for slotted drain pipes,
d	Depth of channel,
d	Depth of flow in gutter,
d_c	Critical depth,
d	Internal diameter of pipe, ft.
D	Diameter of conduit, inside—or maximum span,
D	Depth of corrugation, in.,
D	Minimum cover from top surface of flexible pavement to corrugated steel pipe for airplane wheel loads,
D	Horizontal diameter or span of a tunnel,
D	Long span structure height, ft.
D_c	Critical pipe diameter, in.,
D	Delta, tangent angle, corrugation,
DL	Dead load,
E	Modulus of elasticity, lb/in.^{2},
E	Railroad live load, Cooper,
EOS	Equivalent Opening Size, geotextiles,
f	Friction factor,
f	The rate of infiltration at a specific period of time,
FF	Flexibility factor,
FS	Factor of safety for buckling,
f_a	Allowable wall stress, lb/in.^{2},
f_c	Compressive stress, lb/in.^{2},
f_c	Minimum rate of infiltration,
f_c	Buckling stress, lb/in.^{2},
f_o	Initial rate of infiltration,
f_u	Minimum specified tensile strength, lb/in.^{2},
v	Poisson's ratio,
g	Gravitational acceleration,
h	Height of fill over pipe,
H	Drop of weir notch, ft.
H	The difference in elevation between the most remote point on the basin and the outlet,
H	Head, ft.
H	Height of soil over the top of a tunnel,

Symbol	Definition or Use
h_o	Tailwater depth (HW),
H_e	Critical head,
He	Head, entrance loss,
He	Increment of head above the critical head, ft.
Hf	Head, friction loss,
Hv	Velocity head,
H. HC	Height of cover,
HW	Headwater depth,
H20	Highway live load,
i	Intensity, rainfall, in./hr
i	Transverse slope,
I	Imperviousness, relative,
I	Moment of inertia, in.^{4}/unit of width,
I	Intensity, in./hr,
ia	Intensity before peak rainfall,
ib	Intensity after peak rainfall,
k	Long span entrance coefficient,
k	Rate of decrease in rate of infiltration, f, per unit of time,
K	Soil stiffness factor; load factor,
K	Constant equal to l/S_d,
K	Conveyance,
ke	Coefficient of head loss at entrance,
kdg, kp	Coefficients based on long span inlet type,
ko	Outlet loss coefficient,
l	Length of pipe, ft.
l	Length of opening, ft.
L	Length of weir notch, ft.
L	Maximum length of travel of water, ft.
L	Length of culvert, ft.
L_1, L_2, L_3	Lengths used for live load pressure distribution calculations for pipe arches, in.,
L'	Adjusted value for length,
LL	Live load,
LA	Actual length,
LR	Length with no carryover,
m	Long span entrance coefficient,
n	Roughness factor,
n	Storm frequency,

Symbol	Definition or Use
n	Coefficient of roughness of a gutter
n'	Actual value of Manning's n,
N	Circumferential bolt space (= 3π or 9.6 in),
P	Pressure, external load,
P	Corrugation pitch, in. (5 x 1 in. corrugation),
P	The external load on tunnel liner,
PE	Collapse pressure,
P_c	Pressure acting on soil at pipe arch corners, lb/ft^2,
P_{cr}	Critical pressure, lb/in.^{2},
P_l	The vertical load at the level of the top of the tunnel liner due to dead load,
P_d	Design pressure, liner plate,
P_e	Rainfall excess equal to gross rainfall minus evaporation, interception and infiltration,
p_i	π = 3. 1416,
pH	Hydrogen ion concentration,
P_v	Design pressure, lb/ft^2,
P_v	Design pressure, ring compression,
Φ	Diameter,
Φ	Index of recharge based on constant rate of infiltration,
Q	Discharge, ft^3/sec (peak, volume rate of flow, or quantity reaching a drain),
Q_D	Total flow,
Q_O	Flow in a gutter, ft^3/sec,
r	Ratio of time before the peak intensity occurs to total time duration,
r	Radius of gyration,
R	Resistivity, electrical,
R	Hydraulic radius,
R	Ratio of rise to span,
R	Radius of conveyor cover,
R	Radius of curvature in hook bolt,
R	Radius of pipe, in.,
R_b	Radius of bottom (plates),
R_c	Radius of corner (plates),
R_s	Radius of side (plates),
R_t	Radius of top (plates),
R_1	Long-span inside radius,
R_2	Long-span inside radius,

Symbol	Definition or Use
s	Hydraulic gradient of gutter,
S	Span of arch or pipe arch (or maximum horizontal diameter of any shaped structure),
S	Slope (of ground, channel, invert), ft/ft,
S	Slope, equal to H/L where H is the difference in the elevation between the most remote point on the basin and the outlet, ft/ft,
S	Side slope,
S	Section modulus, in^3,
S_o	Slope, bed (at outlet),
SF	Safety factor (or FS),
S_d	Maximum storage capacity of depression,
t	Time.
T, t	Uncoated thickness of sheet or plate, in.,
T_c	Time of concentration of flow,
TL	Tangent length,
TW	Tailwater depth,
T	Thrust per lineal ft.
T	Width of water surface, ft.
t_a	Time after peak,
t_b	Time before peak,
V	Velocity, mean, ft/sec,
V	Volume of storage at any particular time,
V	Mean velocity of flow, ft/sec,
V_a, V_l	Approach velocity,
V_c	Velocity head,
ΣV	Summation of vertical forces in ring compression calculations,
w	Unit weight of soil, lb/ft^3,
W	Width, conveyor cover,
W	Weight of moist soil,
W, WP	Wetted perimeter,
W	Total weight of soil and live loads over a structure,
WS	Water surface,
X	Distance from neutral axis to outer fiber,
z	Transverse slope reciprocal.

CONVERSION tables

Table C-1 Length

United States System

1 ft	=	12 in.
1 yd	=	3ft
1 mi	=	5280 ft
1 nautical mi	=	1.1515 statute mi

Metric Units

10 mm	=	1 cm
100 cm	=	1 m
1000 m	=	1 km (about 5/8 mi)

Equivalents

1 in.	=	2.54 cm, or 25.4 mm (preferred units)
1 ft	=	0.3048 m
1 statute mi	=	1.60935 km
1 nautical mi	=	1.853 km
1 cm	=	0.39370 in.
1 m	=	3.28 ft
1 km	=	3280.83 ft = 0.62137 mi

Table C-2 Area

United States System

1 ft^2	=	144 in.^{2}
yd^2	=	9 ft^2
	=	1296 in.^{2}
1 acre	=	43,560 ft^2
	=	4840 yd^2
1 mi^2	=	640 acres
	=	1 section of land (U.S.)

Equivalents

1 cm^2	=	0.155 in.^{2}
1 m^2	=	10.76 ft^2
	=	1.196 yd^2
1 km^2	=	0.386 mi^2
1 in.^{2}	=	6.45 cm^2
1 ft^2	=	0.0929 m^2
1 yd^2	=	0.836m^2
1 mi^2	=	2.59 km^2

Table C-3 Volume and Capacity

United States System

1 ft^3 water at 39.1 ° F	=	62.425 Ibs
1 gal (U.S.)	=	231 $in.^3$
1 gal (imperial)	=	277.274 $in.^3$
1 ft^3 water	=	1728 $in.^3$
	=	7.4805 gal (U . S.)
	=	6.2321 gal (imperial)
1 yd^3	=	27 ft^3
	=	46,646 $in.^3$
1 qt	=	2pt
1 gal	=	4 qt
1 gal (U.S.)	=	0.1337 ft^3
	=	0.8331 gal (imperial)
	=	8.345 lbs
1 bbl	=	31.5 gal
	=	4.21 ft^3
1 bu (U.S.)	=	1.2445 ft^3
1 fl oz	=	1.8047 in^3
1 acre ft	=	43,560 ft^3
	=	1613.3 yd^3
1 acre in.	=	3630 ft^3
1 million gal (U .S .)	=	133 ,681 ft^3
	=	3.0689 acre ft
1 ft depth in 1 mi^2	=	27,878,400 ft^3
	=	640 acre ft

Equivalents

1 $in.^3$	=	16.387 cm^3
1 ft^3	=	0.0283 m^3
1 yd^3	=	0.765 m^3
1 cm^3	=	0.061 $in.^3$
1 m^3	=	35.3 ft^3
	=	1.308 yd^3
1 1.	=	61.023 $in.^3$ (approximately 1 qt)
	=	0.264 gal (U.S.)
	=	0.220 gal (imperial)
1 qt (U.S.)	=	0.946 1.
1 gal (U.S.)	=	3.785 1.

Table C-4 Weight

United States System

1 lb	=	16 oz (avdp)
1 ton	=	2000 lbs
1 long ton	=	2240 lbs
1 lb water at 39.1 ° F	=	27.681 in.^{3}
	=	0.016 ft^3
	=	0.1198 gal (U.S.)
	=	0.4536 1.

Equivalents

1 kg	=	2205 lb (avdp)
1 metric ton	=	0.984 long ton
	=	1.102 ton
1 oz (avdp)	=	28.35 gm
1 lb (avdp)	=	0.4536 kg

Table C-5 Pressure

Comparison of Heads of Water with Pressures in Various Units

1 ft water, 39.1 ° F	=	62.425 lbs/ft^2 (psf)
	=	0.4335 lb/in^2 (psi)
	=	0.0295 at.
	=	0.8826 in. mercury at 30° F
	=	773.3 ft air at 32° F and atmospheric pressure
1 ft water at 62° F	=	62.335 lbs/ft^2
	=	0.433 lb/in^2
1 lb water on the in.^{2} at 62° F	=	2.3094 ft water
1 oz water on the in.^{2} at 62° F	=	1.732 in. water
1 at. sea level (32° F)	=	14.697 lbs/in.^{2}
	=	29.921 in. mercury
1 in. mercury (32° F)	=	0.4912 lb/in.^{2}

Table C-6 Flowing Water

cfs	=	ft^3/sec
gpm	=	gal/min
l cfs	=	60 ft^3/min
	=	86,400 ft^3 per 24 hrs
	=	448.83 gal/min (U.S.)
	=	646,317 gal per 24 hrs (U.S.)
	=	1.9835 acre-ft per 24 hrs (usually taken as 2)
1 cfs	=	1 acre-in./hr (approximately)
	=	0.0283 m^3/sec
1 gpm (U.S.)	=	1440 gal per 24 hrs (U.S.)
	=	0.0044 acre-ft per 24 hrs
	=	0.0891 Miners' In., Arizona, California
1 million gal/day (U.S.)	=	1.5472 cfs
	=	3.07 acre-ft
	=	2.629 m^3/min

GENERAL tables

Table G-1 Areas of Plane Figures

Triangle:	Base × ½ perpendicular height $\sqrt{s(s-a)\ (s-b)\ (s-c)}$ s = ½ sum of the three sides a, b and c
Trapezium:	Sum of areas of the two triangles
Trapezoid:	½ sum of parallel sides × perpendicular height
Parallelogram:	Base × perpendicular height
Regular Polygon:	½ sum of sides × inside radius
Circle:	$\pi r^2 = 0.78540 \times \text{dia}^2 = 0.07958 \times \text{circumference}^2$
Sector of Circle:	$\frac{\pi r^2 A°}{360} = 0.0087266\ r^2 A° = \text{arc} \times \frac{1}{2}\ \text{radius}$
Segment of Circle:	$\frac{r^2}{2}\left(\frac{\pi A°}{180} - \sin A°\right)$

Circle of same area as square: diameter = side × 1.12838

Square of same area as circle: side = diameter × 0.88623

Ellipse:	Long diameter × short diameter × 0.78540
Parabola:	Base × ⅔ perpendicular height

Table G-2 Trigonometric Formulas

$$\text{Radius, } 1 = \sin^2 A + \cos^2 A = \sin A \operatorname{cosec} A = \cos A \sec A = \tan A \cot A$$

Sine $$A = \frac{\cos A}{\cot A} = \frac{1}{\operatorname{cosec} A} = \cos A \tan A = \sqrt{1 - \cos^2 A}$$

Cosine $$A = \frac{\sin A}{\tan A} = \frac{1}{\sec A} = \sin A \cot A = \sqrt{1 - \sin^2 A}$$

Tangent $$A = \frac{\sin A}{\cos A} = \frac{1}{\cot A} = \sin A \sec A$$

Cotangent $$A = \frac{\cos A}{\sin A} = \frac{1}{\tan A} = \cos A \operatorname{cosec} A$$

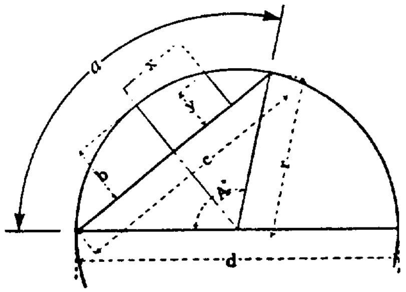
Secant $$A = \frac{\tan A}{\sin A} = \frac{1}{\cos A}$$

Cosecant $$A = \frac{\cot A}{\cos A} = \frac{1}{\sin A}$$

Table G-3 Properties of the Circle*

Circumference of Circle of Diameter 1 = π = 3.14159265
Circumference of Circle = 2 π r
Diameter of Circle = Circumference x 0.31831

Diameter of Circle of equal periphery as Square	= side	x	1.27324
Side of Square of equal periphery as Circle	= diameter	x	0.78540
Diameter of Circle circumscribed about Square	= side	x	1.41421
Side of Square inscribed in Circle	= diameter	x	0.70711



Arc, $a = \dfrac{\pi r A^\circ}{180} = 0.017453\, r A^\circ$

Angle, $A = \dfrac{180^\circ a}{\pi r} = 57.29578 \dfrac{a}{r}$

Radius, $r = \dfrac{4 b^2 + c^2}{8 b}$ Diameter, $d = \dfrac{4 b^2 + c^2}{4 b}$

Chord, $c = 2\sqrt{2 b r - b^2} = 2 r \sin \dfrac{A^\circ}{2}$

Rise, $b = r - \tfrac{1}{2}\sqrt{4 r^2 - c^2} = \dfrac{c}{2} \tan \dfrac{A^\circ}{4} = 2 r \sin^2 \dfrac{A}{4}$

Rise, $b = r + y - \sqrt{r^2 - x^2}$ $y = b - r + \sqrt{r^2 - x^2}$ $x = \sqrt{r^2 - (r + y - b)^2}$

π = 3.14159265, log = 0.4971499

$\dfrac{1}{\pi}$ = 0.3183099, log = $\bar{1}.5028501$

π^2 = 9.8696044, log = 0.9942997

$\dfrac{1}{\pi^2}$ = 0.1013212, log = $\bar{1}.0057003$

$\sqrt{\pi}$ = 1.7724539, log = 0.2485749

$\sqrt{\dfrac{1}{\pi}}$ = 0.5641896, log = $\bar{1}.7514251$

$\dfrac{\pi}{180}$ = 0.0174533, log = $\bar{2}.2418774$

$\dfrac{180}{\pi}$ = 57.2957795, log = 1.7581226

*From Carnegie's "Pocket Companion."

Table G-4 Standard Gages for Sheet and Plate Iron and Steel (Black)

Established by Act of Congress, July 1, 1893 (With revisions, 1945)

Number of Gage	Approximate Thickness: Fractions of an Inch, Wrought Iron*	Approximate Thickness: Decimal Parts of an Inch, Wrought Iron*	Approximate Thickness: Decimal Parts of an Inch, Steel†	Approximate Thickness: Milli-meters, Steel†	Weight: per Square Foot in Ounces Avoir-dupois	Weight: per Square Foot in Pounds Avoir-dupois	Weight: per Square Meter in Kilo-grams
000	3-8	.375	.3587	9.111	240	15.0	73.24
00	11-32	.34375	.3288	8.352	220	13.75	67.13
0	5-16	.3125	.2989	7.592	200	12.50	61.03
1	9-32	.28125	.2690	6.833	180	11.25	54.93
2	17-64	.265625	.2541	6.454	170	10.625	51.88
3	1-4	.25	.2391	6.073	160	10.0	48.82
4	15-64	.234375	.2242	5.695	150	9.375	45.77
5	7-32	.21875	.2092	5.314	140	8.75	42.72
6	13-64	.203125	.1943	4.935	130	8.125	39.67
7	3-16	.1875	.1793	4.554	120	7.5	36.62
8	11-64	.171875	.1644	4.176	110	6.875	33.57
9	5-32	.15625	.1495	3.797	100	6.25	30.52
10	9-64	.140625	.1345	3.416	90	5.625	27.46
11	1-8	.125	.1196	3.038	80	5.0	24.41
12	7-64	.109375	.1046	2.657	70	4.375	21.36
13	3-32	.09375	.0897	2.278	60	3.75	18.31
14	5-64	.078125	.0747	1.897	50	3.125	15.26
15	9-128	.0703125	.0673	1.709	45	2.8125	13.73
16	1-16	.0625	.0598	1.519	40	2.5	12.21
17	9-160	.05625	.0538	1.367	36	2.25	10.99
18	1-20	.05	.0478	1.214	32	2.0	9.765
19	7-160	.04375	.0418	1.062	28	1.75	8.544
20	3-80	.0375	.0359	0.912	24	1.50	7.324

Notes:

By Act of Congress, the gage numbers are based on the weight per square foot in ounce (sixth column) and not on thickness.

* The thickness given in the Congressional table is for wrought iron and not for steel.

† The thickness for steel is from tables compiled by American Iron and Steel Institute, November 1942 based on 41.82 pounds per square foot per inch thick.

• Example: A 16 gage sheet of either wrought iron or steel weighs 40 ounces per square foot. The wrought iron is approximately .0625 inch thick whereas the steel is .0598 inch thick.

GENERAL index

The scope of this book can best be determined by the Table of Contents on page i. The chapters and prime references are shown in bold face. Tables are indicated by T followed by chapter, table number and page number (T1.1, 41). Items listed in the **Glossary**, page 589, are partly cross-referenced to this **General Index**.