Coachella Valley Water District
Pipe Materials for Pressurized Pipeline Projects

1.0 Introduction

This document provides technical information and design guidelines for selecting and designing pressurized pipeline projects within the service area of the Coachella Valley Water District (CVWD) using the pipe materials that have been approved by CVWD.

2.0 Pipe Material Selection Overview

All the generally available materials commonly used for pressurized pipe installations are suitable for use in CVWD's system; however, CVWD has narrowed the range of options to the materials discussed below. Since the specific nature of a given project can dictate the preferred material(s) for a particular installation, each pipe material, lining, and coating should be evaluated carefully and ultimately determined by the Engineer of Record depending on the anticipated installation challenges and conditions of service from both an operations *and* a maintenance point of view. The pipe materials for consideration based upon CVWD's native soil conditions and pressure applications include:

- Flexible Pipes
  - Thermoplastic-based materials:
    - Polyvinyl chloride (PVC) and
    - High-density polyethylene (HDPE)
  - Ductile iron pipe (DIP)
  - Spiral-welded steel pipe (SWSP)
- Rigid Pipes
  - None

The following sections will focus on the above materials presenting a qualitative overview of the characteristics for each pipe material and a summary of the pipe materials options. Each proposed pipe material is evaluated based on available sizes, exterior protective coatings (if required), and interior protective linings (if required). In the event that special conditions require the evaluation of other materials not discussed herein, those materials will need to be addressed on a case-by-case basis by CVWD Engineering.

While this document presents an overview of the approved pipe materials for consideration for projects located in CVWD's service area, further evaluation and development of the site specific standard technical specifications and details of the installation are the responsibility of the Engineer of Record for the proposed improvements.

3.0 Flexible Pipes

Flexible pipes derive essentially all of their external load-carrying capacity from the interaction of the pipe with the embedment soils. During the installation and trench consolidation processes these types of pipes will deflect slightly to accommodate these actions resulting in a small deformation of the geometry that in turn creates a state of static equilibrium. Therefore, a flexible pipe used in a pressurized pipe application must be designed to not only sustain the internal pressure(s) anticipated but also the unpressurized external loading condition. Flexible pipe materials contained herein include thermoplastic pipes, ductile iron pipe (DIP), and spiral-welded steel pipe (SWSP). CVWD currently utilizes these flexible pipe materials in the following applications:

- PVC is used for sewer force mains, canal water irrigation laterals and non-potable water pipelines
- HDPE pipe is used for sewer force mains, canal water irrigation laterals and in special crossing situations (i.e., directional bores) for domestic water and non-potable water
- DIP is used in domestic water and non-potable water (canal water and/or recycled wastewater)
- Spiral-welded large diameter steel pipe is used for non-potable water transmission

3.1 Thermoplastic Pipe Types

Thermoplastic materials include a variety of plastics that can be softened by heating and hardened by cooling through temperature ranges that are specific to each plastic. Thermoplastic pipe products considered appropriate for CVWD projects are polyvinyl chloride (PVC) pipe and high density polyethylene (HDPE) pipe.

3.1.1 Polyvinyl Chloride (PVC)

PVC pipe used within CVWD's system shall be manufactured in accordance with the latest versions of the AWWA C900 or AWWA C905 standards as applicable. Per the direction provided in the applicable sections of CVWD's Development Design Manual (DDM) and this document, the Engineer of Record must be sufficiently skilled in the design parameters of a PVC piping system in a pressure application to provide a finished piping system is cost-effective to install, operate and maintain for the specified design and/or service life period required.

AWWA C900 and C905 PVC Pipe Materials

C900 PVC pipe is manufactured in diameters from 4-inch through 12-inch. Because it is used primarily for water distribution mains, it is produced to three standard pressure classes; 100 psi, 150 psi, and 200 psi. Table 1 below shows the classes to be used by the Engineer of Record for this pipe material by line size and application.

C905 PVC pipe is manufactured in diameters from 14-inch through 42-inch. It is produced in two pressure classes; 165 psi and 235 psi. Again, Table 1 indicates the classes to typically be used by the Engineer of Record for this pipe material by line size and application.

Table 1. Standard Pressure Classes of C900/C905 PVC Pipe by application and line size

Application	Size Range, inches	AWWA Designation	Pressure Class, psi	Dimension Ratio (DR)
Force main	4 - 12	C900	150	18
Force main	14 - 42	C905	165	25
Irrigation line	4 - 12	C900	165	25
Irrigation line	14 - 42	C905	165	25

A General Design Note on Surge Pressures in any Flexible Pipes

Working pressures in full-flowing liquid transport piping systems such as sewer force mains and irrigation water supply lines are not solely hydrostatic pressures. Varying demand for the liquid transport relating to pump and/or valve operations in the piping system will result in working pressure fluctuations. The non-steady state operation of these systems typically results in some measure of cyclical surging. When the cyclic surging is anticipated to be of a high frequency or magnitude, the design of the piping system should not be based primarily on the hydrostatic pressure rating assigned to the pipe product.

Any pipe material can experience fatigue failure when exposed to severe cyclic surging that exceeds the design limits of that material. Fatigue failure of PVC pipes in a sewer force main application may be governed by severe cyclic surging. If the cyclic surges are not controlled or designed out of the system, then all the pipes, fittings, and appurtenances must be designed with sufficient allowance for cyclic surging in order to prevent fatigue failure of the system.

Surge pressures are generated in pressure pipes as a direct result in the change in velocity of the flowing fluid. The magnitude of these pressure variances is a function of the piping material's modulus of elasticity (E) and the rate at which the velocity change is allowed to occur. Further, surge pressure analysis is not only concerned with positive pressure variances but also the negative pressure variances. Full vacuum can occur when there is a separation of the fluid column in a pipeline; which commonly occurs with a loss in power or rapid closure of an upstream valve. In these instances, thin wall ferrous and low stiffness thermoplastic pipe may be subjected to buckling failure due to this vacuum pressure if not accounted for in the design of the piping system. The likelihood of buckling is even greater when the pipe is somewhat elliptically shaped as a result of its installation.

There are two categories of damage that can occur from surge events:

1) catastrophic failure of the pipeline system or equipment caused by exceeding the pipe's working pressure rating or,

2) fatigue failure of the pipeline, supports, instrumentation, equipment and components due to recurring surge events beyond the capabilities of the installed pipe material.

Pressure transient events are an important part of the design process; especially with PVC thermoplastic pipe materials. The Engineer of Record is encouraged to review the paper *"The Need for Comprehensive Transient Analysis of Distribution Systems"* by Jung et al in the January 2007 issue of the AWWA Journal.

For most flexible pipes such as steel, ductile iron, and thermoplastic based materials, a combined loading analysis is not necessary. For these materials, the pipe is designed as if external loading and internal pressure were acting independently. As the pressure design usually controls; a pipe thickness or strength is chosen first on the basis of the internal pressure performance requirements and then an engineering analysis is made to insure that the chosen pipe will withstand the external loads of the installation site.

PVC Pipe for Sewer Force Mains, Canal Water Irrigation Laterals and Non-Potable Water Pipelines

Force main design for PVC pipes shall be performed in accordance with the Uni-Bell Technical Report written on this application; UNI-TR-6 PVC Force Main Design. PVC pipes possess two life resources; static and dynamic (for hydrostatic and cyclic). These funds are separate and independent of each other. The cyclic life pressure fund is a critical parameter if the number of cycles is very large or if the magnitude of the surge pressure is high. To select the appropriate PVC pipe for a proposed application the following steps as presented in UNI-TR-6 shall typically be taken:

1. Determine the years of service required of the application.

2. Determine the average pressure and the surge pressure amplitude in the proposed pipeline system.

3. Calculate the average hoop stress and stress amplitude for the proposed class of pipe.

4. Use the graph shown in Figure 1 to determine the number of cycles that will cause failure.

5. Based on this data and the projected cyclic rate of the proposed application, calculate the expected system lifetime.

6. If the calculated life is not sufficient, return to step 3 and use a higher class of pipe.

The Engineer of Record shall employ, where practical, appurtenant works to control the cyclic stresses and/or rate of occurrence to acceptable levels using the preferred class of PVC material stated in Table 1.

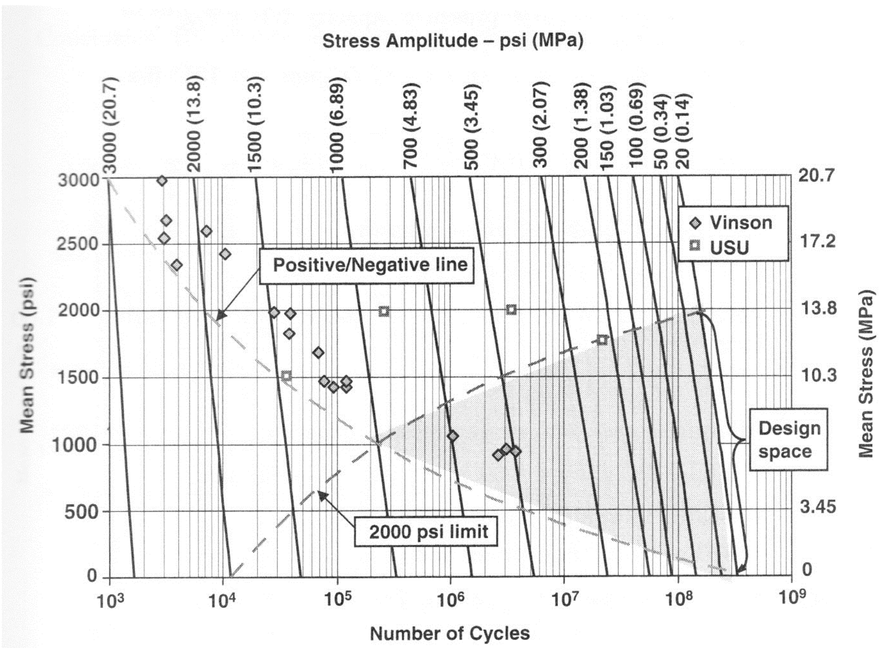


Figure 1 - PVC Design Space for cyclic pressure applications

3.1.2 High Density Polyethylene Pipe (HDPE)

Solid wall HDPE pipe is suitable for domestic water, non-potable water, canal water irrigation laterals and sewer force main system piping. Piping within the CVWD service area shall be manufactured to the AWWA standard C906 in either 40 or 50 foot standard lengths. Pipe sizes for material manufactured to this standard range between 4-inches and 63-inches in diameter; and dimension ratios (DR) between 7 and 41. The required Hydrostatic Design Stress (HDS) shall be determined by the Engineer of Record using the design methodology outlined by the Plastic Pipe Institute (PPI) in their Handbook of PE Pipe (Design of PE Systems - Chapter 6).

HDPE pipes differ in how they handle surge pressures. Because of HDPE's relatively low stiffness to that of the other flexible materials the peak value of a surge pressure is significantly lower than that of the PVC and metallic pipes mentioned herein. The second difference is the material's fatigue resistance. HDPE pipes have essentially an unlimited service life when the positive surge pressure variation plus the sustained pressure is less than or equal to 2.0 times the pressure rating of the selected pipe wall thickness for occasional pressure surges (1.5 times the pressure rating when the application has recurring pressure surges).

CVWD Engineering subscribes to the design guidance provided in Chapter 6 of the Handbook of PE Pipe regarding the engineering approaches that are to be taken for addressing the occasional surge pressures, recurring surge pressures and negative pressure design conditions. It is the responsibility of the Engineer of Record to address these needs both from a standpoint of the proposed system's operation and the pipe material's long-term performance capabilities.

3.2 Thermoplastic Pipe Installation Requirements

All the thermoplastic pipe materials discussed above should follow the installation design guidance given in ASTM D2321; *Standard Practice for Underground Installation of Thermoplastic Pipe for Sewers and Other Gravity-Flow Applications* (or a similar standard that is applicable to the particular flexible pipe material being installed).

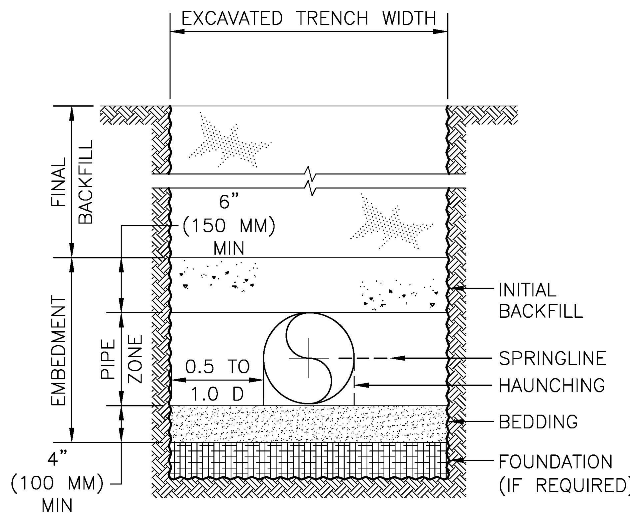


Figure 2

The soil support for a buried pipe installation can be expressed as the elastic modulus (E') of the soil or the composite constrained soil modulus (M_s). M_s is a function of the constrained modulus of the backfill material and the native soil, M_{sb} and M_{sn}, as well as the trench width. For pipe installations in soft native soils where M_{sn} is lower than M_{sb}, the composite modulus, M_s, will be lower than the backfill modulus, M_{sb}. ASTM D2321 advises that a minimum width of embedment material is required to ensure that adequate embedment stiffness is developed to support the pipe. Under poor native soil (MH, CH, OL, OH, PT; or CL, or any soil beginning with one of these symbols, with <30% retained on the #200 sieve), if the native soil is able to sustain a vertical cut as is depicted in Figure 2, this minimum embedment width is recommended to be 0.5 pipe diameters (O.D.) on either side of the pipe as shown. If the native soil cannot sustain a vertical cut, the minimum embedment width shall be at least one pipe diameter on either side of the pipe. Also, the trench width shall not be less than that required per Section 6.3 of ASTM D2321.

Soil modulus is a measure of the soil's stiffness which, in turn, is a measure of the level of structural support it can provide. In buried piping design it is sometimes referred to as the "constrained" soil modulus which calls to the designer's attention that as the height of the pipe cover increases so does the apparent stiffness of the embedment soil. This is due to the confining pressure exerted by the fill above in conjunction with the trench sides and bottom. In the case where the embedment soil is different than the native soil material, a composite modulus must be derived to reflect the actual in place performance of the pipe embedment.

The embedment material shall typically be Class II granular material or Class I crushed angular granular material as specified in the Table 2 below. Select Class III materials may be suitable for shallow depth sewers when installed above the water table. When coarse and open-graded material is placed adjacent to a finer material, fines may migrate into the coarser material under the action of a hydraulic gradient from groundwater flow into the trench. Significant hydraulic gradients may arise in the pipeline trench during construction when water levels are being controlled by various pumping or well-pointing methods or after construction when permeable under-drain or embedment materials act as a drain under high groundwater levels. Field experience shows that migration can result in significant loss of pipe support and continuing deflections that may exceed design limits. The gradation and relative size of the embedment and adjacent materials must be compatible in order to minimize migration. In general, where significant groundwater flow is anticipated, avoid placing coarse, open-graded materials, such as SC I, below or adjacent to finer materials unless methods are employed to impede migration. Where incompatible materials must be used, they must be separated by filter fabric designed to last the life of the pipeline to prevent wash-away and migration. The filter fabric must completely surround the bedding and pipe zone backfill material and must be overlapped over the top of the pipe embedment zone.

Table 2. Soil classification by material type

Soil Class	Soil Type	Description of Material Classification
I	—	Manufactured angular, granular material, ¼ to 1 ½ inches size, including materials having regional significance such as crushed stone or rock, broken coral, crushed slag, cinders, or crushed shells
II	GW	Well graded gravels and gravel-sand mixtures, little or no fines, clean
	GP	Poorly-graded gravels and gravel-sand mixtures, little or no fines, clean
	SW	Well-graded sands and gravelly sands, little or no fines, clean
	SP	Poorly-graded sands and gravelly sands, little or no fines, clean
III	GM	Silty gravels, gravel-sand-silt mixtures
	GC	Clayey gravels, gravel-sand-clay mixtures
	SM	Silty sands, sand-silt mixtures
	SC	Clayey sands, sand-clay mixtures
IV	ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands. Liquid limit (LL) less than 50
	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. LL less than 50
	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts. Liquid limit 50 or greater
	CH	Inorganic clays of high plasticity, fat clays. Liquid limit 50 or greater
V	OL	Organic silts and organic silty clays of low plasticity. Liquid limit less than 50
	OH	Organic clays of medium to high plasticity. Liquid limit more than 50
	PT	Peat, muck, and other highly organic soils

Symbols for soil type are according to the Unified Soil Classification Designation, ASTM D2487

When supports such as trench sheeting, trench jacks, trench shields or boxes are used, the installer must make sure that the support of the pipe and its embedment is maintained throughout the installation process. Sheeting must be sufficiently tight so as to prevent washing out of the trench wall from behind the sheeting. Unless otherwise directed by the Engineer of Record, sheeting driven into or below the pipe zone should be left in place to preclude any loss of support of the foundation and embedment materials. When the installer chooses to cut off the top of the sheeting, this cut should be made at least 1.5 feet above the crown of the pipe. Rangers, whalers, and braces must be left in place to support the cut off sheeting.

Movable trench wall supports should not be used below the top of the pipe zone unless approved methods are used for maintaining the integrity of the embedment material; this is true for both rigid and non-rigid materials. Before moving supports, place and compact embedment to sufficient depths to ensure protection of the pipe. As supports are moved, finish placing and compacting the embedment. All voids must be filled immediately upon removal of these supports.

When excavating while depressing the groundwater level, the installer must make sure that the temporary surface of the water is at least 5.0 feet below the bottom of the cut at all times to prevent washout from behind the sheeting or sloughing of the exposed trench walls. Control of the water level below the bottom of the cut should be maintained before, during, and after pipe installation, and until the embedment is installed and sufficient backfill has been placed to prevent flotation of the pipe. To preclude loss of soil support from the native materials' original measured values, the dewatering methodology must employ techniques that minimize the removal of fines and the creation of voids in the in-situ materials.

The following Table 3 summarizes the degree of compaction which can be obtained for the soil classes defined in Table 2 using the various tools available. Table 4, in turn, gives the Engineer of Record an estimate of the M_{sb} that can be expected when using the various soil classes as an embedment material.

Table 3. Degree of Compaction obtainable by class of material by level of effort applied

Class of Embedment	I	II	III	IV
Material Description	Manufactured Granular Materials	Sand & Gravel Soils - Clean	Mixed – Grain Soils	Fine-Grain Soils
Optimum moisture content range		9 - 12	9 – 18	6 - 30
Soil Consolidation Method	% of Standard Proctor (or Relative) Density Range			
Compact by power tamper or rammer	95-100 (75-100)	95-100 (80-100)	95-100	90-100
Densification by portable vibrators	80-95 (60-75)	80-95 (60-80)	80-95	75-90
Consolidate by saturation		80-95 (60-80)		
Hand placing	60-80 (40-60)	60-80 (40-60)		
Hand tamping		60-80 (50-60)	60-80	60-75
Dumping	90-95 (85-90)	60-80 (50-60)	60-80	60-75

Table 4. Approximate values of M_{sb} at various vertical stress levels (cover depths)

Soil Class	Vertical Stress Level psi (ft)	Compaction, % maximum Standard Proctor Density				
		Dumped psi	100 psi	95 psi [ft]	90 psi [ft]	85 psi [ft]
I	1 (1.2)	2000	2350	2000		
I	5 (6)	2600	3450	2600		
I	10 (12)	3000	4200	3000		
I	20 (24)	3450	5500	3450		
II	1 (1.2)		2350	2000	1275 [1085]	470 [330]
II	5 (6)		3450	2600	1500 [1275]	520 [365]
II	10 (12)		4200	3000	1625 [1380]	570 [400]
II	20 (24)		5500	3450	1800 [1530]	650 [455]
III	1 (1.2)			1415 [708]	670 [335]	360 [180]
III	5 (6)			1670 [835]	740 [370]	390 [195]
III	10 (12)			1770 [885]	750 [375]	400 [200]
III	20 (24)			1880 [940]	790 [395]	430 [215]
IV	1 (1.2)			530 [159]	255 [77]	130 [39]
IV	5 (6)			625 [188]	320 [96]	175 [53]
IV	10 (12)			690 [207]	355 [107]	200 [60]
IV	20 (24)			740 [222]	395 [119]	230 [69]

Burial depths, or vertical stress levels, are for a soil density of 120pcf. Reduced M_{sb} values below groundwater table in brackets.

In 2000, AASHTO adopted new values for soil stiffness for backfill materials used for thermoplastic pipe. The modifications included changing the soil design parameter from the modulus of soil reaction, E', to the constrained soil modulus, M_s. This change, based on the work of McGrath (1998), resulted in the above presented (Table 4) design values of the constrained modulus which shows that M_s increases with the depth of fill which reflects the increased confining pressure. This is a well-known soil behavior. At moderate depths of fill the values of M_s are close to the E' values proposed earlier by Amster Howard (1997, 1996).

Table 5. Approximate values of M_{sn} for native granular and cohesive soils

Granular Soils		Cohesive Soils		Modulus
Blow count	Description	q_u tons/ft^2	Description	M_{sn} [psi]
> 15	Compact	> 2.0	Very stiff	5000
8 - 15	Slightly compact	1.0 – 2.0	Stiff	3000
4 - 8	Loose	0.5 – 1.0	Medium	1500
2 - 4	Loose	0.25 – 0.50	Soft	700
1 - 2	Very loose	0.125 – 0.25	Very soft	200
0 - 1	Very, very loose	0 – 0.125	Very, very soft	50

Blow counts per standard penetration test [ASTM D1586)

Table 5 above summarizes the approximate values for the constrained soil modulus of the native soil or M_{sn}. Geotechnical sampling or in place resistance testing is used to measure the stiffness of the existing soils.

3.3 Ductile Iron Pipe

AWWA C151 Ductile Iron Pipe (DIP) is to be used for domestic water and non-potable water systems in sizes ranging from 8-inches to 42-inches in diameter; CVWD specifically prefers to use 8, 12, 18, 24, 30, 36, and 42-inch diameter pipe. For non-potable water applications the pipeline shall be a minimum of 12-inches in diameter. All DIP and fittings shall be cement mortar lined per AWWA standard C104.

The required wall thickness shall be determined by the Engineer of Record using the American National Standard for the Thickness Design of Ductile Iron Pipe (ANSI/AWWA C150/A21.50). As mentioned earlier herein (3.1.1), this type of pipe material is designed separately to withstand external loads and internal pressure. The design procedure at a minimum shall include the following steps:

1. Design for internal pressures (hydrostatic plus surge pressure allowance).

    a. The net thickness required for internal pressure is calculated using the equation for hoop stress. The design internal pressure is equal to the safety factor of 2.0 times the sum of the working pressure plus the surge pressure allowance. The standard surge allowance of 100 psi is typically incorporated into various design aids which work for most applications. However, the Engineer of Record is responsible for determining whether the anticipated actual surge pressure is at or above this value when using these design aids.

2. Design for bending stress due to external loads (earth loads plus any applicable live loads).

    a. The net wall thickness required for the external load is based on two design considerations; limitation of ring bending stress and ring deflection. The design equation for ring bending stress shall be as shown in the referenced design standard for the pipe laying condition (installation trench configuration) selected by the Engineer of Record.

3. Select the larger resulting net wall thickness for steps 1 and 2.

4. Ensure that 0.08-inch service allowance is included in the pipe thickness in step 3 (typically included per ANSI/AWWA C150/A21.50)

5. Check for allowable deflection performance.

    a. The maximum allowable ring deflection for cement mortar lined DIP is 3% of the outside diameter of the pipe (5% for flexible linings). Deflection limits of 3% for CML pipe will provide a factor of safety of 2.0 against failure of the liner. The deflection performance shall be calculated using the equation in the referenced standard for the laying condition (installation trench configuration) selected by the Engineer of Record.

6. Add an allowance to the wall thickness for casting tolerances as presented in the referenced design procedure.

7. Select the pressure class required to provide the minimum wall thickness determined from step 6.

3.4 DIP Installation Requirements

Ductile iron pipe is to be installed using the same philosophy as for all flexible pipes. The Engineer of Record can use either; a) the installation standard ANSI/AWWA standard C600 or, b) the flexible trench design referenced in section 3.2 herein for thermoplastic pipes to guide the design of the installation trench detail.

The minimum cover for pipes installed within CVWD system shall be as stated in the Development Design Manual (DDM) Section 5, Table 5.5 for the applicable site specific condition of the proposed installation. All pipe and fittings shall be installed in a polyethylene encasement per ANSI/AWWA C105/A21.5.

3.5 Spiral-Welded Steel Pipe (SWSP)

3.5.1 Spiral-Welded Steel Pipe (in accordance with AWWA C200 and ASTM A139)

Spiral-welded steel pipes (SWSP) are an allowable material for non-potable water transmission lines within CVWD's system. Lining and coating systems to be employed shall be determined by the

Engineer of Record during the design process based upon the site specific conditions found by engineering analysis and the required design and/or service life specified by CVWD.

- Lining – Water Service
  - Cement mortar per AWWA C205
  - Epoxy per AWWA C210
  - Polyurethane per AWWA C222
- Coating – Exterior
  - Epoxy per AWWA C210
  - Polyurethane per AWWA C222
  - Tape wrap system per AWWA C214

The basic criterion for the design of a steel pipe is resistance to internal pressure. Once that criterion has been met, the Engineer of Record should verify that the resulting wall thickness is adequate for the other performance criteria; external loads, handling, and buckling (external pressure).

The Engineer of Record shall refer to AWWA's Manual M11 (Steel Water Pipe: A Guide for Design and Installation) for the design and installation of SWSP. The design procedure presented therein is quite similar to that of the previous section's DIP. The wall thickness is first calculated based on the maximum sustained internal operating pressure, and then calculated based on the larger of the maximum operating plus transient pressure, or the field test pressure.

Because of the relatively more flexible nature of the SWSP, the minimum required wall thickness based on transportation and handling before and during its installation should be limited by a D/t (nominal diameter/thickness) ratio of 240 for shop applied cement mortar linings; and D/t ratio of 288 for flexible shop applied linings or field applied cement mortar linings.

SWSP for CVWD projects shall utilize the rolled groove O-ring gasketed joint; i.e. bell and spigot.

3.6 Spiral-Welded Steel Pipe Installation Requirements

Spiral-welded steel pipe (SWSP) shall be installed in a flexible pipe trench condition as per section 3.2 described herein. Additionally, the Engineer of Record shall use the guidance given in Chapters 12 and 13 of the referenced AWWA Manual M11 regarding the supplementary design data and installation details, transportation considerations, installation procedures, and testing procedures of the SWSP to insure that all aspects of the proposed installation are adequately addressed in the detailed plans and specifications prepared to execute the proposed project.

4.0 CVWD Specific Guidelines for Design

It is the intent of this section of the document to review and provide general design guidance to the design engineering community on the currently available pressure pipe materials within normal shipping distances of CVWD. Each project undertaken in the CVWD service area poses different challenges and no one document can be expected to prescribe a pipe type without an independent and thorough engineering evaluation of site specific conditions which must be satisfied.

4.1 Pipe Material Selection

The Engineer of Record shall follow a materials selection approach that considers the anticipated project's site specific conditions and utilize the material(s) that are best suited for those conditions. Table 6 below, has been developed to provide a general idea of how currently available pipe materials compare against each other given size, native soil class, and proposed depth of bury. Alternative trench designs can extend the range of applicability, but require a specialist engineer trained is soil-structure interaction to design the proper trench configuration.

Table 6. Pipe Selection – Maximum Cover Depth versus Soil Stiffness Class

Material	Size Range inches	Max Depth (ft)	Native Soil Conditions			
			Class II	Class III	Class IV	Class V
PVC – C900	4 to 12	30	30	20	Alt. Design	Alt. Design
PVC – C905	14 to 42	30	30	20	Alt. Design	Alt. Design
HDPE	8 to 42	20	20	15	Alt. Design	Alt. Design
DIP - CML	8 to 42	30	30	25	Alt. Design	Alt. Design
SWSP - CML	24 to 42	30	30	25	Alt. Design	Alt. Design

4.2 Factor of Safety (FS)

The design of the trench installation detail is governed by the load which a pipe must support in a specific application which is a function of the class of bedding that is specified, the native materials stiffness, and the inherent strength of the chosen pipe material(s). For flexible pipes the Engineer of Record inserts the factor of safety (FS) directly into the equations used for estimating the in situ deflection and strain performance of the selected material(s). It is the Engineer of Record's responsibility to set an appropriate FS based on the design parameters used in his/her design calculations and the construction difficulties anticipated in the field associated with those choices. CVWD recommends that in no case should the Engineer of Record produce an installation detail that provides for a FS of less than 2.5 in the deflection or ring bending performance limit states.

Highlighted in the following box are the components that should go into the decision making process regarding selecting an appropriate site specific FS for the design of the proposed improvements.

Factor of Safety (FS)

The assignment of a FS is done by accounting for:

- the magnitude of damages related to any potential loss of life or property damage,
- the relative cost of increasing or decreasing the FS,
- relative change in the probability of a failure by changing the FS,
- the reliability of the soil data,
- construction tolerances,
- changes in the soil properties due to the anticipated construction operations, and
- the accuracy (or approximations used) in developing the design/analysis methods.

CVWD's minimum recommended factor of safety is 2.5 for H/D ≥ 2, or 3.0 for H/D < 2

(Where H is the height in feet of the fill over the pipe and D is the O.D. of the pipe in feet)

4.3 Flexible Pipe Deflection

All flexible pipe materials are subject to some deflection during the pipe's installation and the subsequent consolidation of the embedment zone material(s) in the first six to nine months. The trench design for these pipe materials will need to take into account the potential for this and the need to keep the deflection below the selected pipe material's known performance limits. The Engineer of Record's calculations should clearly demonstrate that this design limit state will be preserved with the proposed design trench detail given the level of effort that has been used to determine the conditions present on the project site.

4.4 Pipe Embedment Material

The pipe embedment material is selected by the Engineer of Record to properly support the proposed type of pipe material according to its needs and the native soils found to be present on the project site. It is very important to assess constructability and CVWD's minimum recommended FS, which is 2.5 for H/D ≥ 2, or 3.0 for H/D < 2 (with "H" being the height of the fill over the pipe and "D" being the O.D. of the pipe; both these variables are in feet).

Side support is very necessary for flexible pipes. The vertical loads on a flexible pipe and the resultant horizontal movement develops a passive soil resistance that varies depending on the soil type and the degree of compaction of the pipe zone backfill material. The historical parameter used to characterize the soil's stiffness (resistance) in the design of flexible pipe is the modulus of soil reaction, or *E*'. In 2000, AASHTO adopted new soil stiffness values for the backfill materials around flexible pipe reflecting how the soil's resistance increases with the depth of fill due to the increasing confining pressure (e.g. effective vertical pressure on the soil plane at the top of the pipe). This new soil term was designated as M_s, or the constrained soil modulus. To determine M_s for a buried pipe, separate M_s values for the native soil (M_{sn}) and the pipe backfill surround (M_{sb}) must be determined using either the information gathered from a site soils survey, or Tables 3 and 4 contained in section 3.2; and then combined using the soil support combining factor as shown below in Table 7 ($M_s = S_c \times M_{sb}$).

Table 7. Values for the soil support combining factor Sc

M_{sn}/M_{sb}	B_d/D = 1.25	B_d/D = 1.50	B_d/D = 1.75	B_d/D = 2.0	B_d/D = 2.5	B_d/D = 3.0	B_d/D = 4.0	B_d/D = 5.0
0.005	0.02	0.05	0.08	0.12	0.23	0.43	0.72	1.00
0.01	0.03	0.07	0.11	0.15	0.27	0.47	0.74	1.00
0.02	0.05	0.10	0.15	0.20	0.32	0.52	0.77	1.00
0.05	0.10	0.15	0.20	0.27	0.38	0.58	0.80	1.00
0.1	0.15	0.20	0.27	0.35	0.46	0.65	0.84	1.00
0.2	0.25	0.30	0.38	0.47	0.58	0.75	0.88	1.00
0.4	0.45	0.50	0.56	0.64	0.75	0.85	0.93	1.00
0.6	0.65	0.70	0.75	0.81	0.87	0.94	0.98	1.00
0.8	0.84	0.87	0.90	0.93	0.96	0.98	1.00	1.00
1.0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
1.5	1.40	1.30	1.20	1.12	1.06	1.03	1.00	1.00
2	1.70	1.50	1.40	1.30	1.20	1.10	1.05	1.00
3	2.20	1.80	1.65	1.50	1.35	1.20	1.10	1.00
≥5	3.00	2.20	1.90	1.70	1.50	1.30	1.15	1.00

B_d is the trench width; D is the nominal diameter of the pipe.

Initial Backfill placement is designed to protect the new pipe during the backfilling process and to insure that no rocks larger than 6-inches in size will come into contact with the pipe. The level of compaction required is to be commensurate with the material being used and the need to evenly distribute the vertical loading onto the soil plane at the top of the pipe.

The following guidelines can be used by the Engineer of Record as a basis for judgment in determining the suitability of a soil for the embedment material on a particular site. These guidelines, however, are by no means applicable to every field condition that may be encountered.

- Well-graded, angular bedding materials are more stable, allow less movement, and are more resistant to migration when flooded than rounded bedding materials of equal gradation.
- The stability of a bedding material increases as its particle size increases. However, gradations containing particles greater than 1.0 inch become increasingly more difficult to place into the pipe haunch area and may result in uneven support.

- Fine materials are subject to more movement than those of a larger sieve size.
- Sand is suitable as a bedding material in a total sand environment. However, where high or rapidly changing water tables are present in the pipe zone, consideration should be given to the use of an angular bedding material with a geo-fabric material for support. Sand is not an appropriate material for bedding or haunching in a trench cut by blasting or in trenches through hard clay soil.
- Controlled Low Strength Material (50-300 psi) has been shown to be an economic alternative to other bedding materials and classes. It assists in utilizing the inherent strength of the pipe, completely fills the haunch area, and reduces the trench load on a rigid pipe.

4.5 Trench Width Requirements

The minimum and the maximum trench widths should be clearly stated on the project's installation detail. The minimum trench width is necessary to insure that there is adequate room for the selected embedment zone material(s) to be properly placed around the pipe (e.g. under the haunches of the pipe). For flexible pipe materials the minimum width is also important to insure that the selected embedment material(s) will develop the resistance stiffness required to support the pipe.

4.6 Trench Bottom's Bearing Capacity

All pipe manufacturers call for the bottom of the excavated trench to be a "firm and unyielding surface". This means that the design of the installation trench needs to support the vertical loading that the Engineer of Record's calculations indicate is going be borne by the pipe. If the strength of the soil at the bottom of the trench is estimated to be, or found during the installation of the pipe, to be less than this loading, the trench must be excavated an additional depth to find this strength in the soil or to reduce the loading on the soil at the bottom of the trench to agree with the native soil's in situ capabilities. This is typically accomplished by placing a coarse rock or other suitable foundation material between this elevation and the bottom of the pipe bedding layer to provide the firm and unyielding surface for the pipe's bedding.

The bottom of the trench shall be shown to be accurately graded to provide uniform bearing and support for each section of pipe at every point along its entire length excluding that at the bell area. For purposes of quantifying what the minimum level of support is, a handheld penetrometer or other suitable tool shall be utilized to measure the amount of resistance in the soil at that depth. A reading of between 1.0 and 2.0 tons/sf (tsf) will be considered as passing this requirement. Where the bottom of the trench reads less than 1.0 tsf, the Contractor should be directed to excavate the trench in additional 6-inch increments until this minimum bearing capacity is obtained. The maximum additional excavation in any one area shall be limited to 18 inches when the minimum bearing capacity at this depth measures at least 0.5 tsf. If this does not occur, the Contractor shall request input from the Engineer of Record as to how to proceed. The foundation material shall consist of 1-½" inch minus rock of a gradation to prevent migration of the native materials into its matrix (e.g. Caltrans Class 2 aggregate base, 1-1/2" size).

4.7 Transient Pressure Analysis

The transient pressure analysis must be performed to a level of sophistication that is commensurate with the complexity of the proposed pressure pipe system or system addition. Simple surge allowances are not sufficient to protect pipes from early failure due to fatigue. It is the responsibility of the Engineer of Record to perform an analysis of the anticipated critical operating conditions to correctly identify the performance parameters of the pressure pipe system for the required service life.

Figure 3 below graphically represents the results of an analysis made of a 10-inch force main composed of two different pipe materials; DIP for the first 720 m (2,362 feet) and PVC pipe for the remaining 930 m (3,051 feet). This computer generated analysis illustrates how the transient pressures impacted the anticipated operating conditions which, in turn, influenced the selection and the design of the pipe materials for this force main application. Notice the impact on the energy grade line (shown as rated HGL in the figure) from the use of the two different materials. Comparing the EGL with the HGL is an excellent means of identifying potential problem areas such as an area of potential cavitation (shown as "vapour" pressure issues on the figure).

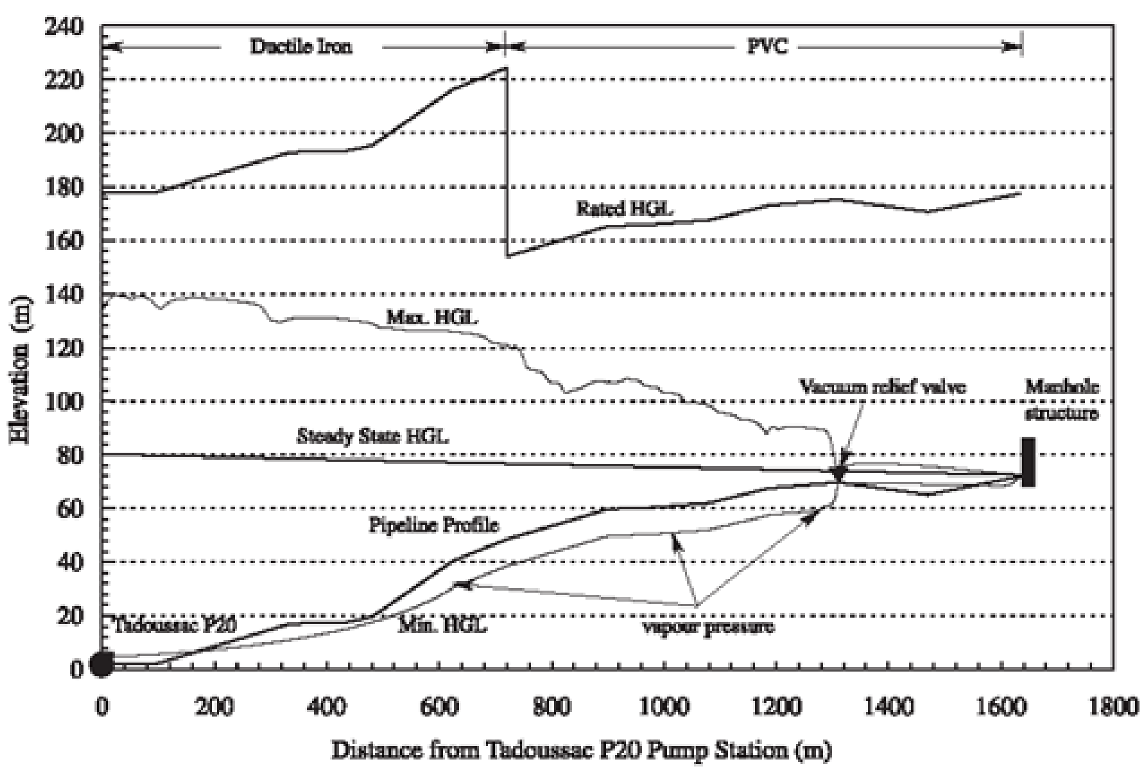
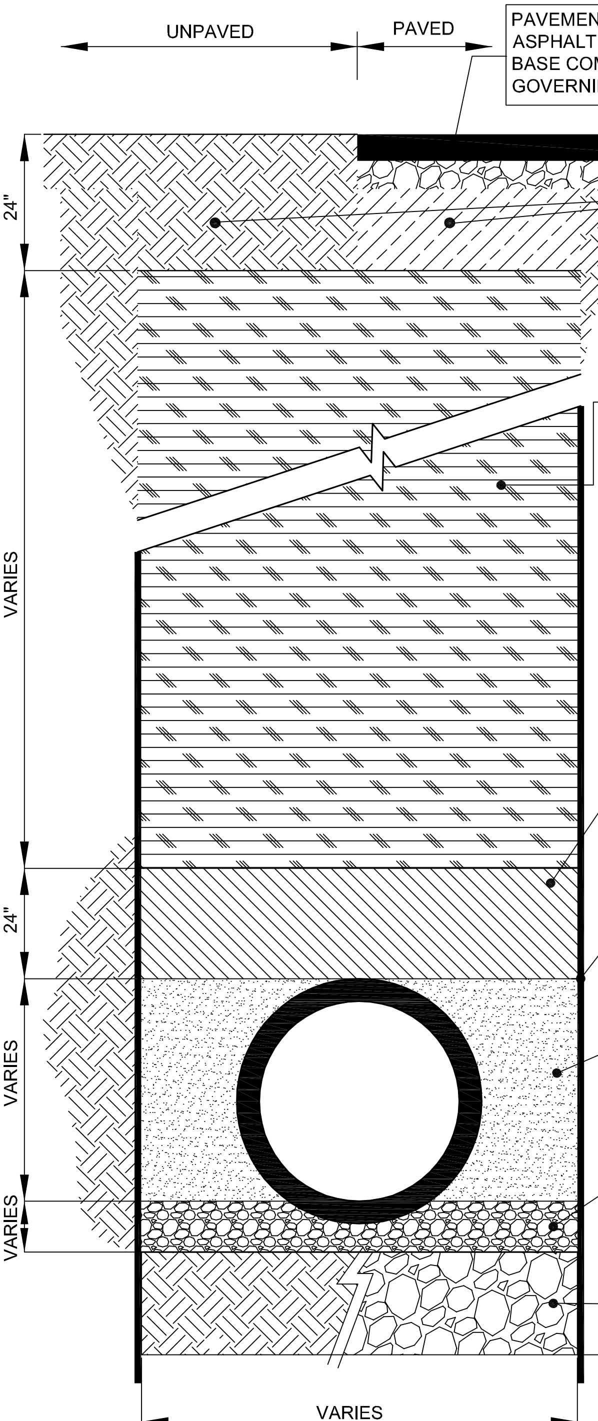


Figure 3 - Transient Internal Pressure analysis results of a 10-inch force main sewer.

4.8 Trench Detail Guideline

The trench detail shown in the following page has been developed to serve as a guide to CVWD's minimum expectations of the design engineering community as to what is required to produce a proper installation detail for projects undertaken within their jurisdiction. This is NOT a standard detail. The components are presented in an informational format to communicate with the Engineer of Record where and how the controlling elements of his or her design analysis are to be communicated to the pipeline installation contractor.



PAVEMENT REPLACEMENT SHALL BE A MINIMUM OF 4" OF HOT-MIX ASPHALT CONCRETE PAVEMENT OVER 8" OF CLASS II AGGREGATE ROAD BASE COMPACTED TO 95% COMPACTION, OR AS PER THE LOCAL GOVERNING AUTHORITY; WHICHEVER IS GREATER.

THE NATIVE SOIL OR SELECT MATERIAL SHALL BE PLACED IN 8" LIFTS (LOOSE) AND COMPACTED TO AT LEAST 95% COMPACTION PER ASTM D1557. COMPACTION SHALL BE VERIFIED BY TESTING .

THE NATIVE SOIL MATERIAL MAY BE USED IN THE TRENCH BACKFILL PROVIDED IT'S FREE FROM UNSUITABLE MATERIAL INCLUDING MATERIAL THAT CANNOT BE STABILIZED BY COMPACTION, MATERIAL GREATER THAN 6" IN SIZE, VEGETATION, TRASH, CONSTRUCTION DEBRIS, HIGHLY EXPANSIVE CLAYS, HIGHLY ORGANIC SOILS AND CONTAMINATED SOILS. THE FINE SILT AND CLAYEY MATERIALS MAY BE USED PROVIDED THEIR MOISTURE CONTENT CAN BE CONTAINED WITHIN THE RANGE THAT ALLOWS FOR THEIR PROPER COMPACTION. THE NATIVE SOIL SHALL BE PLACED IN 8" LIFTS (LOOSE) AND COMPACTED TO AT LEAST 90% COMPACTION PER ASTM D1557, MODIFIED PROCTOR TEST (APPROXIMATELY 95% COMPACTION PER ASTM D698, STANDARD PROCTOR TEST). COMPACTION SHALL BE VERIFIED BY TESTING. IF NATIVE SOIL IS DEEMED NOT SUITABLE, USE BACKFILL PER SSPWC SECTIONS 200-1.2 AND 306-1.3 AND/OR AS SPECIFIED BY THE GEOTECHNICAL ENGINEER.

THE INITIAL BACKFILL ON TOP OF THE PIPE EMBEDMENT ZONE SHALL CONSIST OF A LAYER OF SELECT MATERIAL PLACED TO A DEPTH OF 24" BY 8" LIFTS. THE SELECT MATERIAL SHALL BE COMPACTED TO 90% COMPACTION PER ASTM D1557, MODIFIED PROCTOR TEST (APPROXIMATELY 95% COMPACTION PER ASTM D698, STANDARD PROCTOR TEST). COMPACTION SHALL BE VERIFIED BY TESTING.

THE DESIGN ENGINEER-OF-RECORD MUST CONSIDER THE NEEDS OF THE PIPE MATERIAL THAT PRODUCED THE DESIGNED TRENCH CONFIGURATION AND ANY POTENTIAL CONSTRUCTABILITY ISSUES FOR THE SITE-SPECIFIC CONDITIONS OF THE PROJECT AND MUST CLEARLY COMMUNICATE/DESCRIBE THE CRITICAL PARTS OF THE DESIGN IN THE CONTRACT DOCUMENTS SO THAT THE CONTRACTOR IS ABLE TO PROPERLY DESIGN SHEETING AND SHORING AND UNDERSTANDS ITS IMPACTS ON THE INTEGRITY OF THE PIPE EMBEDMENT WHEN THE SHEETING/SHORING IS EXTRACTED. IN SOME CASES, IT MAY BE NECESSARY TO LEAVE THE SHEETING/SHORING IN PLACE FROM THE BOTTOM OF THE TRENCH TO THE TOP OF THE PIPE (AT A MINIMUM).

THE PIPE EMBEDMENT ZONE IS TO BE DESIGNED BY THE DESIGN ENGINEER-OF-RECORD TO TAKE INTO CONSIDERATION AN APPROPRIATE FACTOR OF SAFETY (2.5 MINIMUM). THE DESIGN NEEDS TO TAKE INTO CONSIDERATION THE NEEDS OF THE PIPE MATERIAL AND ANY CONSTRUCTABILITY ISSUES FOR THE SITE SPECIFIC CONDITIONS.

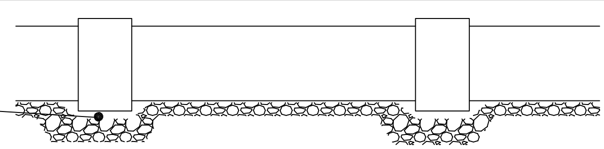
PIPE BEDDING MATERIAL TO BE DESIGNED BY THE DESIGN ENGINEER-OF-RECORD MUST TAKE INTO CONSIDERATION THE DESIGN REQUIREMENTS FOR THE SELECTED PIPE MATERIAL(S).

THE DESIGN ENGINEER-OF-RECORD, BASED ON GEOTECHNICAL INFORMATION OBTAINED FOR THE PROJECT, SHALL EVALUATE AND CONFIRM THE PREDICTED BEARING CAPACITY OF THE NATIVE SOIL AT THE BOTTOM OF THE TRENCH MEETS THE REQUIRED PIPE BEARING CAPACITY OF THE DESIGN TRENCH. WHEN THE BOTTOM OF THE TRENCH DOES NOT MEET THE REQUIRED BEARING CAPACITY (IS NOT FIRM ENOUGH TO SUPPORT THE PIPE BEDDING FOR THE DESIGN LOADING CALCULATED), THE DESIGN ENGINEER-OF-RECORD SHALL DETERMINE THE DIMENSIONS OF A GRAVEL FOUNDATION THAT IS REQUIRED AND THE TYPE OF GRAVEL AND IT'S GRADATION TO BE USED TO ACHIEVE THIS REQUIREMENT. THE TRENCH FOUNDATION SHALL BE ACCURATELY GRADED TO PROVIDE UNIFORM BEARING SUPPORT FOR EACH SECTION OF PIPE AT EVERY POINT ALONG ITS LENGTH.

A MINIMUM AND MAXIMUM TRENCH WIDTH (MEASURED AT THE TOP OF THE PIPE), SHALL BE SPECIFIED. THESE TWO PARAMETERS DEFINE THE WORKING ROOM REQUIRED FOR THE PIPE EMBEDMENT MATERIAL PLACEMENT AND SOIL LOADING USED IN THE PIPELINE INSTALLATION DESIGN OF THE TRENCH.

WHEN A PIPE IS TO BE INSTALLED BELOW THE GROUND WATER TABLE, THE GROUND WATER SHALL BE LOWERED TO AT LEAST 5 FEET BELOW THE LOWEST EXCAVATED TRENCH ELEVATION. THE TRENCH FOUNDATION SHALL BE STABLE WITH NO PONDED WATER, MUD OR MUCK AND SHALL MEET THE REQUIREMENTS NOTED ABOVE.

BELL HOLES SHALL BE PROVIDED AT EACH JOINT WITH A MINIMUM CLEARANCE OF 1" FROM THE BEDDING MATERIAL.



COACHELLA VALLEY WATER DISTRICT

PRESSURIZED PIPELINE TRENCH DETAIL DESIGN GUIDELINES

DATE: OCTOBER 2014

J:\ENGCAD\DETAILS\Details_Preliminary\TRENCH DETAIL 2013\PIPELINE TRENCH DETAIL DESIGN GUIDELINE11/13/2013

Attachments:

1. "The need for comprehensive transient analysis of distribution systems" by Jung et al, AWWA Journal, January 2007
2. PVC Force Main Design. Uni-Bell Technical Report UNI-TR-6
3. ASTM D2321 – 11 Standard Practice for Underground Installation of Thermoplastic Pipe for Sewers and Other Gravity-Flow Application

Many surge analysis and design rules have evolved over time to help utilities cope with the complexity of transient phenomena. These rules have been widely applied to simplify analysis by restricting both the number and difficulty of the transient cases that need to be evaluated. On further reflection, however, the implicit assumption that elementary and conservative rules are a valid basis for design has often been shown to be questionable and sometimes dangerous. Indeed, many published guidelines are so misleading and so frequently false that they should only be used with extreme caution, if at all. This article specifically reviews a number of guidelines or suggestions found in various AWWA publications for water hammer analysis and provides a set of warnings about the misunderstandings and dangers that can arise from such simplifications. The authors conclude that only systematic and informed water hammer analysis can be expected to resolve complex transient characterizations and adequately protect distribution systems from the vagaries and challenges of rapid transients.

The need for comprehensive transient analysis of distribution systems

BY BONG SEOG JUNG,
BRYAN W. KARNEY,
PAUL F. BOULOS,
AND DON J. WOOD

A distribution system is not a single entity but rather comprises a complex network of pipes, pumps, valves, reservoirs, and storage tanks that transports water from its source or sources to various consumers. It is designed and operated to consistently and economically deliver water in sufficient quantity, of acceptable quality, and at appropriate pressure. Huge amounts of capital will continue to be spent on the design of new distribution systems and the rehabilitation of existing systems in both developing and developed countries. The magnitude of the needs is a challenge even to visualize: in the United States alone, some 880,000 mi of unlined cast-iron and steel pipes are estimated to be in poor condition, representing an approximate replacement value of $348 billion (Clark & Grayman, 1998).

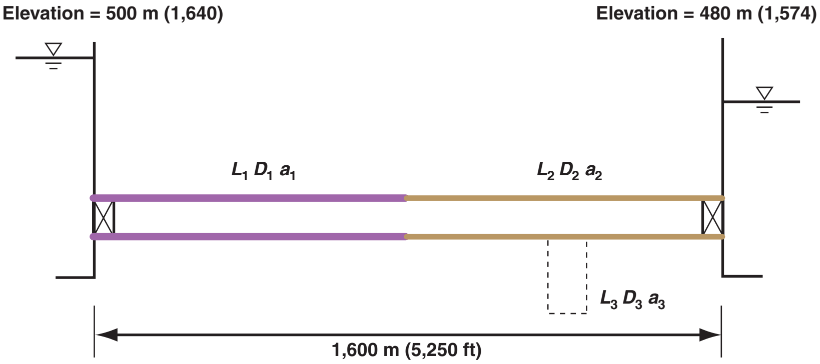
BACKGROUND

Transients. Of the many challenges that face water utilities, one critical but too-often-forgotten issue is protecting the system from excessive transient or water hammer conditions. Surge analysis is essential to estimate the worst-case scenarios in the distribution system (Boulos et al, 2005). In essence, transients occur whenever flow conditions are altered, for they are the physics of change, bringing “news” of any adjustment throughout the network. However, transients are most severe when rapid changes occur, such as those resulting from power failure, emergency valve operations, or firefighting. These changes are generally characterized by fluctuating pressures and velocities and are critical precisely because pressure variations can be of high magnitude, possibly large enough to break or damage pipes or other equipment or to greatly disrupt delivery conditions.

Transient regimes in a distribution system are inevitable and will normally occur as a result of action at pump stations and control valves. Regions that are particularly susceptible to transients are high elevation areas, locations with either low or high static pressures, and regions far removed from overhead storage (Friedman, 2003).

Firefighting demands. Distribution systems sometimes are called on to deliver large flow demands at adequate pressures for firefighting. Although these fire demands occur infrequently, they may constitute a highly constraining factor in pipeline design. Design procedures therefore should evaluate the ability of the system to meet firefighting demands at all relevant hydrant locations. Even though the occurrence of simultaneous fires at all possible locations is not realistic, a variety of firefighting demand patterns must still be considered. Under transient conditions, the designer must anticipate both the establishment of firefighting flows and their ultimate curtailment, a process that often unfolds rapidly in time and can create significant transient pressures, particularly if fire crews receive little specific training or instruction.

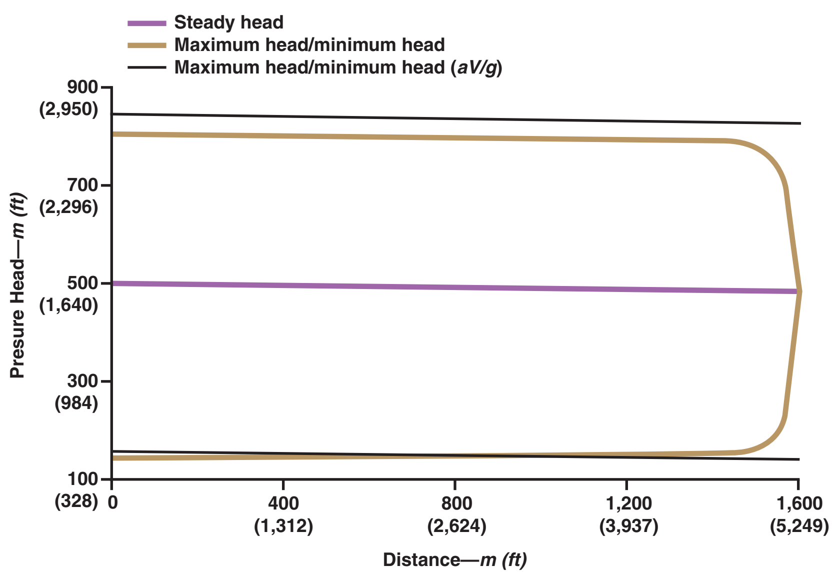
FIGURE 1 Case study of single-pipeline system



a—*wave speed,* D—*diameter,* L—*length*

Three systems are considered: a pipeline with uniform properties, a pipeline with reflection points created by changes in properties, and a pipeline with an attached dead-end pipe segment.

FIGURE 2 Transient response for upstream valve closure



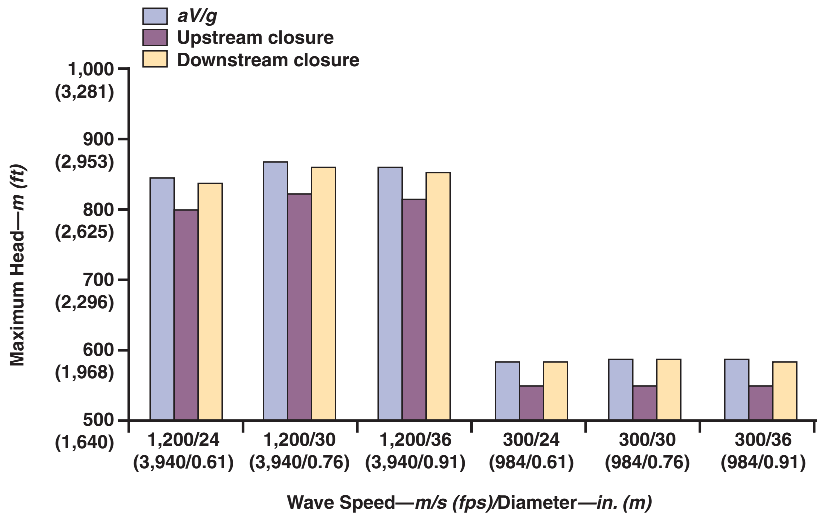
a—*wave speed in m/s,* aV/g—*Joukowski surge head,* D—*diameter in in.*

Properties are a/D *(1,200/24 in.).*

Water quality considerations. A more recently highlighted motivation for conducting a surge analysis arises from water quality considerations. One of the challenges in managing distribution system water quality is that contaminants can intrude into pipes through leaks from reduced- or negative-pressure transients. In reality, all pipeline systems leak, and hydraulic transients occur more or less continuously in distribution systems, so it is not surprising that low-pressure transients introduce a considerable risk of drawing untreated and possibly hazardous water into a pipeline system (Fernandes & Karney, 2004; McInnis, 2004; Karney, 2003). In fact, soil and water samples were recently collected adjacent to drinking water pipelines and then tested for occurrence of total and fecal coliforms, *Clostridium perfringens, Bacillus subtilis,* coliphage, and enteric viruses (Karim et al, 2003). The study found that indicator microorganisms and enteric viruses were detected in more than 50% of the samples examined.

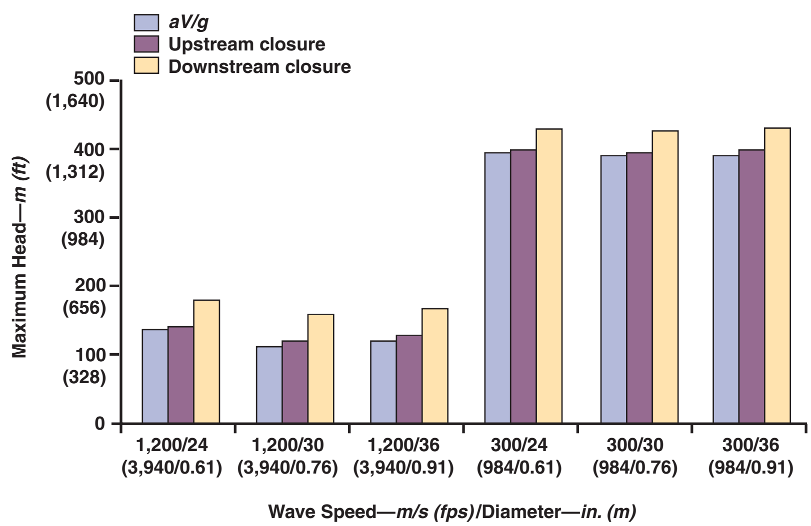
These and other results suggest that during negative- or low-pressure situations, microorganisms can enter the

FIGURE 3 Maximum head of uniform pipeline



aV/g—*Joukowski surge head*

FIGURE 4 Minimum head of uniform pipeline



aV/g—*Joukowski surge head*

distribution system directly through pipeline leaks. For these reasons, the designer should not overlook the effects of water hammer or pressure surges in the design and operation of the distribution system or the evaluation of either system performance or ultimate system cost.

Design obstacles. In spite of the importance of water hammer, the remaining and obviously troublesome problem is the relative complexity of the required computer modeling and engineering analysis. The governing equations describing the transient flow represent a set of nonlinear partial differential equations with sometimes sophisticated boundary conditions. In addition, the hydraulic devices are complex, performance data are difficult to obtain and sometimes poorly understood, and pipeline systems themselves are subject to a variety of operating conditions and requirements. To make matters worse, the physical characteristic of the pulse wave propagation is frequently hard to visualize or interpret, even for the analyst accustomed to transient phenomena (Karney & McInnis, 1990).

This complexity of both transient phenomena and analysis has at times induced engineers to use simplified design procedures. Many simplified guidelines have been published in the past and can be found in various AWWA literature, e.g., Manual M11, *Steel Water Pipe—A Guide for Design and Installation* (AWWA, 2004); Manual M23, *PVC [polyvinyl chloride] Pipe—Design and Installation* (AWWA, 2002); and C403-00, *Selection of Asbestos–Cement Transmission Pipe, Sizes 18 in. Through 42 in. (450 mm Through 1,050 mm*; AWWA, 2000). However, any limited approach should carefully consider a fundamental question: "Are the simplifications both conservative and reasonable?" Unfortunately, the a priori assumption of design that some rudimentary and conservative system can be found is questionable. This article identifies several of these misconceptions or limitations of simplified rules and describes the general weakness and danger of the simplified designs for water hammer. Case studies illustrate the potential for erroneous application by comparing a comprehensive analysis with a simplified one.

REVIEW OF RULES

Guideline examples. To set the stage for more detailed discussion, it is useful to briefly summarize a few specific guidelines found in the AWWA literature. The AWWA literature has not been singled out for its particularly extreme views; rather, it is readily at hand and is typical of a large body of relatively accessible and widely dispersed literature. Seven examples from the guidelines are given, followed by a discussion of the basis—and sometimes the danger—of each articulated position.

Surge pressure. The pressure rise for instantaneous closure is directly proportional to the fluid velocity at cutoff and to the velocity of the predicted surge wave but is independent of the length of the conduit (AWWA, 2004; 2000). Thus, the relation used for analysis is simply the well-known Joukowski expression for sudden closures in frictionless pipes:

$$h = \frac{aV}{g} \qquad (1)$$

in which h is the surge pressure, V is the velocity of water in the pipeline, a is the wave speed, and g is the gravitational acceleration.

Wave speed. Pressure waves are established that move through the pipeline system at rates of 2,500–4,500 fps (760–1,370 m/s), with the exact rate depending primarily on the pipe wall material. The velocity of the wave is the same as the velocity of sound in water, modified by physical characteristics of the pipeline and is estimated by the following equation (AWWA, 2000):

$$a = \frac{V_s}{\sqrt{1+\frac{kd}{Ee}}} \qquad (2)$$

in which k is the modulus of compression of water, d is the internal pipe diameter, E is the modulus of elasticity of the pipe, e is the pipe wall thickness, and V_s is the velocity of sound in water.

Assumption of uniform properties. When the flow rate is changed in a time greater than zero but less than or equal to 2 L/a s in which L is the pipeline length, a is the wave speed, and s is the time in seconds, the magnitude of the pressure rise is the same as with instantaneous closure, but the duration of the maximum value decreases as the time of closure approaches 2 L/a s (AWWA, 2004). The thinking here is that the time it takes for a water hammer wave to travel the length of the system and back (i.e., 2 L/a) is the minimum time needed for the possibly mitigating effect of boundary conditions at the far end of the system to be experienced.

Maximum pressure. The maximum pressure at the control valve exists along the full length of the line with instantaneous closure and for slower rates moves up the pipe a distance equal to $L - (Ta/2)$ in which T is the closing time and then decreases uniformly (AWWA, 2004).

Assumption of surge pressure independence from pipeline profile. The surge pressure distribution along the conduit is independent of the profile or ground contour of the line as long as the total pressure remains above the vapor pressure of the fluid (AWWA, 2004).

Valve closing and maximum pressure rise. For valve closing times greater than 2 L/a s, the maximum pressure rise is a function of the maximum rate of change in flow with respect to time, dV/dt (AWWA, 2004).

Pipe design and selection for pressure surges. To design or select a pipe for occasional pressure surges, the following approach is sometimes recommended:

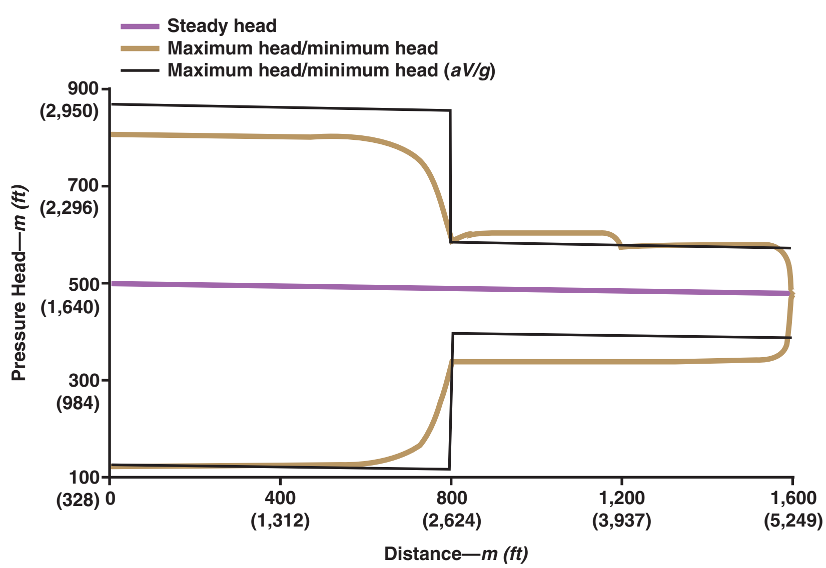
$$WPR = STR - (V \times P'_s) \qquad (3)$$

in which WPR is the working pressure rating in psi, STR is the short-term rating of the pipe in psi, V is the actual system velocity in fps, and P'_s is 1-fps surge pressure in psi. The STR is calculated by applying a factor of safety ($SF' = 2.5$) to the short-term strength (STS) of PVC pipe as in Eq 4:

$$STR = STS/SF' \qquad (4)$$

These levels should be considered to be the design surge capacity limits for PVC pressure pipe manufactured to AWWA standards for a transmission main application (AWWA, 2002).

FIGURE 5 Transient response for upstream valve closure



a—wave speed in m/s, aV/g*—Joukowski surge head,* D*—diameter in in.*

Properties are a_1/D_1 *(1,200/30) and* a_2/D_2 *(300/30).*

Where simple rules can break down. Of course, these examples from AWWA transient guidelines do have considerable basis in fact. For example, the origin of the first rule is the famous fundamental equation of water hammer, which is also called the Joukowski relation. The origins of this relationship are somewhat complex, as Tijsseling and Anderson (2004) have pointed out. This relation equates the change in head in a pipe to the associated change in fluid velocity. However, such a relation is applicable only under restricted circumstances. When the required conditions are met, the simple relationships are often as powerful and accurate as they are easy to determine. However, most published guidelines, such as those found in various AWWA literature, are primarily applicable to simple changes such as a sudden flow stoppage in a single pipeline. Although the initiating trigger is often assumed to be conservative in that sudden stoppage is a severe event, the degree or lack of conservatism is never evaluated. Thus the overall effect of the approach may not be conservative.

To demonstrate some of the difficulties for many of the AWWA rules summarized in the previous sections, the following discussion is intended to raise a number of warnings about where these guidelines might be misleading and thus lead to a poor basis of design. After these general concepts, specific systems are described to demonstrate some of these warnings more precisely.

Surge pressure: how the rule breaks down. The guidelines suggest that the pressure rise for instantaneous closure is directly proportional to the fluid velocity and is independent of conduit length; the associated initial upsurge (aV/g) is often referred to as the potential surge. In general, it might be reasonable to use this potential surge concept in a short pipeline fed from a reservoir and controlled by a valve. However, in a long pipeline, the total drop in hydraulic grade line over the pipe length for the initial flow may be greater than the potential surge (Wylie & Streeter, 1993). Because only part of the flow is stopped by the first compression wave and then the flow is stopped totally at the valve, an increase in stored mass continues—a phenomenon known as line packing. Thus, the pressure continues to rise, the pipe wall expands, and the liquid continues to be compressed after the initial flow stoppage. More generally, the guideline relation attributes no significant role either to friction loss (which can either dissipate or accentuate the surge pressure) or the system's profile. In fact, it is often important to remember that most pumping systems move water uphill, so that the "natural" flow direction is negative; thus, after a power failure, the flow tends to reverse if a check valve is not installed to prevent this, and the potential change in velocity is often much greater than the initial velocity. This and many other circumstances can create an actual surge much larger than the so-called potential surge.

Huge amounts of capital will continue to be spent on the design of new distribution systems and the rehabilitation of existing systems in both developing and developed countries.

Wave speed: how the rule breaks down. Wave speed is a function of many fluid and pipe properties (e.g., pipe diameter, thickness, and material; pipe restraint conditions; water density, elasticity, temperature, air, and solids content). Some of these conditions can be accurately assessed, but others can be difficult or uncertain and depend on a complex set of interacting operating conditions. For this reason, an analysis sensitive to uncertainty in the value of the wave speed is an essential component of surge analysis and design work, and variations in the wave speed should be expected and accounted for. The largest estimate of wave speed may not correspond to the greatest actual surge conditions, even though it is clearly associated with the largest potential surge.

Assumption of uniform properties: how the rule breaks down. Simple relationships are available or applicable only for a single uniform pipeline experiencing simple events. Simple relationships do not consider wave reflections from different pipe properties, nor do they allow for the influence of friction. Thus another problem

TABLE 1 System information for a uniform pipeline

a m/s (fps)	*D* in. (m)	*V* m/s (fps)	*aV/g* m (ft)
1,200 (3,940)	24 (0.61)	2.82 (9.25)	344.7 (1,131)
1,200 (3,940)	30 (0.762)	3.01 (9.88)	368.0 (1,207)
1,200 (3,940)	36 (0.914)	2.94 (9.65)	359.4 (1,179)
300 (984)	24 (0.61)	2.82 (9.25)	86.2 (283)
300 (984)	30 (0.762)	3.01 (9.88)	92.0 (302)
300 (984)	36 (0.914)	2.94 (9.65)	89.9 (295)

a—wave speed, *aV/g*—Joukowski surge head, *D*—diameter, *V*—velocity is the implicit assumption of uniform pipeline properties. If a single pipeline has different physical properties (i.e., diameter, pipe material, and wall thickness), a reflected wave will originate from each discontinuity point, producing a different, and sometimes more severe, transient response (Wylie & Streeter, 1993). If the system in question is a network of pipes, the pressure rise is strongly influenced by system topology. The pipes in a system can thus be a source of transient waves or a receiver–transmitter, and these different roles influence the nature of the assessment.

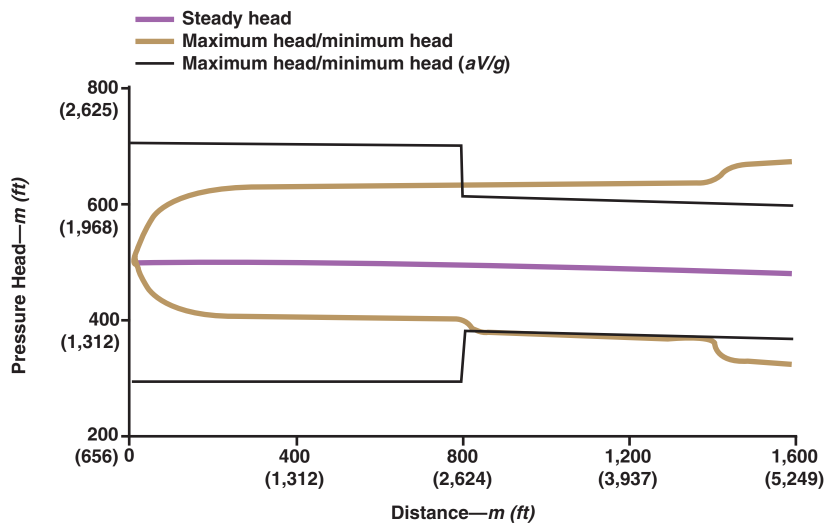
Maximum pressure: how the rule breaks down. For slower rates of change in simple systems, the relationships indicate the maximum pressure travels up the pipe a distance equal to $L - (Ta/2)$ and then decreases uniformly. Only at the extreme end of the pipe are simple rules applicable. Granted, considering relatively sudden changes is an attempt to be conservative, but as is shown later, the rules are not conservative in many cases, nor are sudden changes particularly rare. Modern pumps typically have such small rotational inertia that power failures often generate essentially instantaneous changes; however, the overall system dynamics can create maximum pressures that are significantly greater than those predicted through the potential surge concept.

Assumption of surge pressure independence from pipeline profile: how the rule breaks down. The independence of the profile or ground contour of the line, as well as the characteristic of the hydraulic grade line, can directly influence the pressure heads that occur under surge conditions. Clearly, if the surge pressure is expressed as pressure head (as is appropriate to the stress conditions in the pipe wall), the surge pressure depends on the profile along the pipeline.

Valve closing and maximum pressure rise: how the rule breaks down. For more complex transients, simple rules cannot be applied even for a single uniform pipeline because the reflected waves modify the overall response. Therefore, the calculation of the nature and influence of reflected waves should be included in the analysis and decision process.

Pipe design and selection for pressure surges: how the rule breaks down. Like many other simplified rules, the design for water hammer considers the rapid transient within a single uniform pipeline only. If the surge-induced operation time can be modified (i.e., to more than 2 L/a), the diminished surge pressure may reduce the pipe cost.

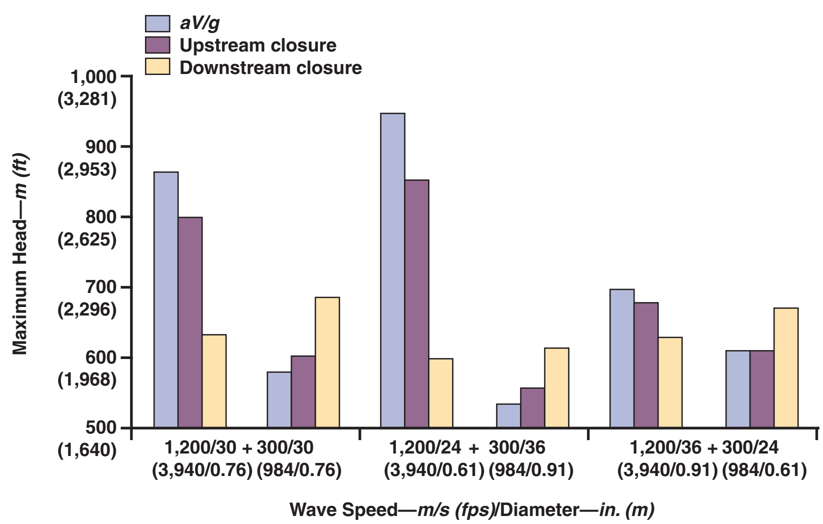
FIGURE 6 Transient response for downstream valve closure



a—wave speed in m/s, aV/g—Joukowski surge head, D—diameter in in.

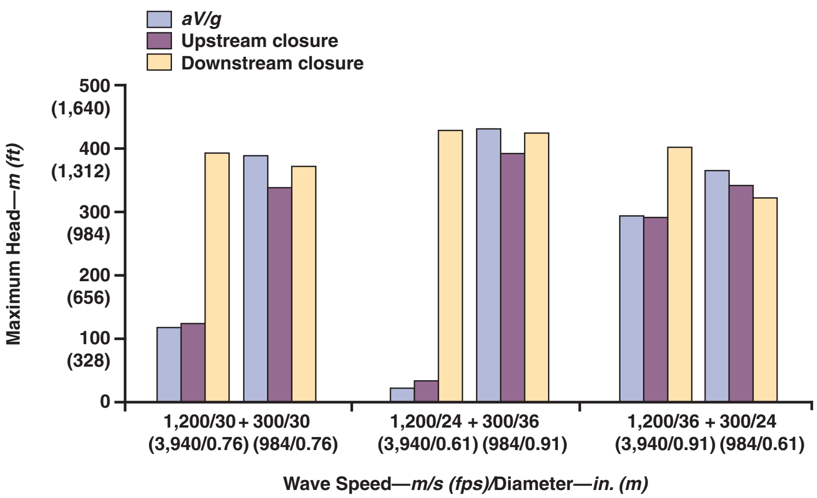
Properties are a_1/D_1 *(1,200/36) and* a_2/D_2 *(300/24).*

FIGURE 7 Maximum head of nonuniform pipeline



aV/g—*Joukowski surge head*

FIGURE 8 Minimum head of nonuniform pipeline



aV/g—*Joukowski surge head*

Because of the dangers of an imprecise water hammer analysis, a large safety factor, often set at 2.5 or higher, is sometimes used in an attempt to cover these contingencies. With the safety factor largely arbitrary, however, a high safety factor can create a twofold problem: (1) the strength might be unreasonably large, creating an unnecessarily expensive system or (2) should the factors of safety be insufficient, the pipe strength might be inadequate, leaving the system vulnerable to water hammer.

To summarize, simple rules such as those found in several AWWA publications ignore the complications of interaction of the different pipe properties in a distribution system. Actual pipes in distribution systems are necessarily connected, and water hammer waves are significantly affected by these connections. At pipe junctions and dead ends, wave reflections and refractions occur, which often magnify or attenuate the surge waves. Moreover, simplified rules cannot simulate a variety of loadings in the quest for the worst-case scenarios in a distribution network. Any reflective practitioner must ask, "What's at stake?" In fact, both overdesign and underdesign can put the system at risk. This risk can take the form of a risk to the pipeline and its associated hydraulic devices, a risk of water contamination, and even a risk to human life. The next section explores and illustrates these claims in more detail.

CASE STUDIES

The purpose of these case studies was to apply and compare comprehensive surge analyses and simplified analyses suggested by the rules published in the AWWA literature. In particular, the studies provide counter-examples to show how and when the simplified rules can break down. Comprehensive water hammer analysis is defined here as the transient analysis that can simulate a head loss resulting from friction and wave reflection from any hydraulic devices or boundary conditions in the system. It can be produced numerically using either the method of characteristics (Wylie & Streeter, 1993) or the wave characteristic method (Boulos et al, 2006; Wood et al, 2005a; 2005b). Indeed, any of these results are reproducible using any number of commercial or in-house water hammer codes.

The case study shown in Figure 1 represents a single pipeline system. The system comprises a pipe connected to two reservoirs with a head difference of 20 m (65.6 ft). The length and Hazen-Williams roughness coefficient for the pipe are 1,600 m (5,250 ft) and 120, respectively. Three pipeline systems are considered: one with uniform properties, one with reflection points created by changes in properties, and one with an attached dead-end pipe segment. The terminal reservoirs, each having a valve with a discharge coefficient (e.g., the valve's *Cv* or *Es* value) of unity, which means that a head loss of 1 m (3.28

TABLE 2 System information for nonuniform pipelines

Pipe 1				Pipe 2			
a *m/s (fps)*	*D* *in. (m)*	*V* *m/s (fps)*	*aV/g* *m (ft)*	*a* *m/s (fps)*	*D* *in. (m)*	*V* *m/s (fps)*	*aV/g* *m (ft)*
1,200 (3,940)	30 (0.762)	3.01 (9.88)	368.0 (1,207)	300 (984)	30 (0.762)	3.01 (9.88)	92.0 (302)
1,200 (3,940)	24 (0.61)	3.71 (12.2)	453.3 (1,487)	300 (984)	36 (0.914)	1.65 (5.41)	50.5 (166)
1,200 (3,940)	36 (0.914)	1.65 (5.41)	201.9 (662.4)	300 (984)	24 (0.61)	3.71 (12.2)	113.3 (371.7)

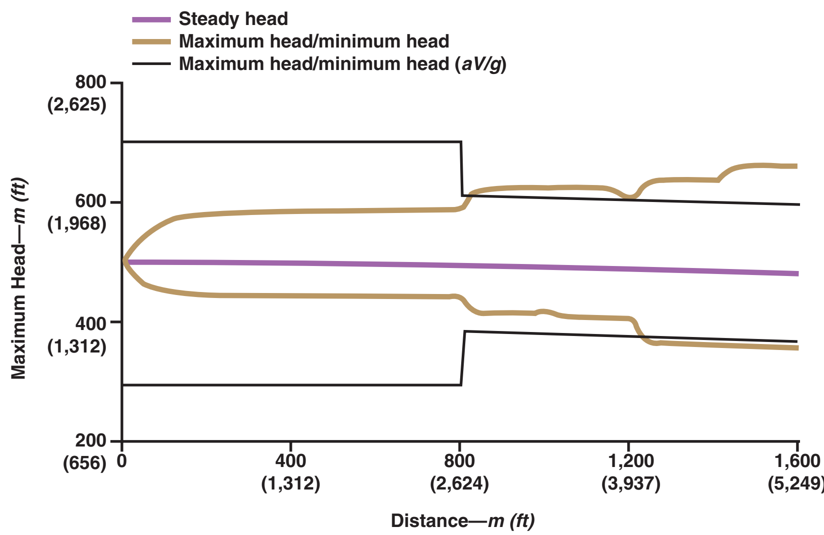
a—wave speed, *aV/g*—Joukowski surge head, *D*—diameter, *V*—velocity ft) occurs when the valve discharges 1 m^3/s (35 cu ft/s; Karney & McInnis, 1992). To introduce transient conditions into this case study, a rapid valve closure (1 s) was chosen.

Uniform pipeline. A uniform single pipeline is first considered. Table 1 shows the wave speed (*a*), diameter (*D*), corresponding velocity (*V*), and Joukowski surge head (*aV/g*). Two values of wave speed—1,200 m/s (3,940 fps) and 300 m/s (984 fps)—were used to represent a rigid pipe (e.g., steel) and an elastic pipe (e.g., PVC). In addition, different pipe diameters—24 in. (0.61 m), 30 in. (0.762 m), and 36 in. (0.914 m)—were used and compared to set the stage for subsequent study into nonuniform pipeline systems.

Figure 2 shows the case of an upstream valve closure. The first downsurge initiated at the upstream valve is similar to the Joukowski downsurge except that the friction loss along the pipeline causes a slightly different slope along the pipe. However, the reflected upsurge from the downstream reservoir results in a significant head difference from the Joukowski upsurge even though the valve closure time (1 s) is fast enough to be classified as a rapid closure (i.e., the valve operation time is less than 2 *L/a*). The reason for this difference is the dissipation of downsurge and upsurge in the downstream reservoir for 1 s. The downsurge from an upstream valve propagates to the downstream reservoir and then is converted into the corresponding upsurge. At the same time, the upsurge interacts with the remaining downsurge, causing some pressure dissipation, which is added to the frictional dissipation in the pipe.

Figures 3 and 4 depict the maximum and minimum pressure head for the uniform pipeline model shown in Table 1 for the different locations (upstream or downstream) of the valve closure. Not surprisingly, the higher wave speed systems have greater upsurge and downsurge pressures than those with lower wave speeds. The two figures also show that the downsurge pressures attributable to the closure of the upstream valve, as already indicated in Figure 2, are similar to the Joukowski surge pressure (*aV/g*); however, the upsurge pressures consistently give a head difference of ~40 m (131 ft) regardless of wave speed and diameter. For the case of the downstream valve closure, the observed trends are exactly opposite those associated with the upstream

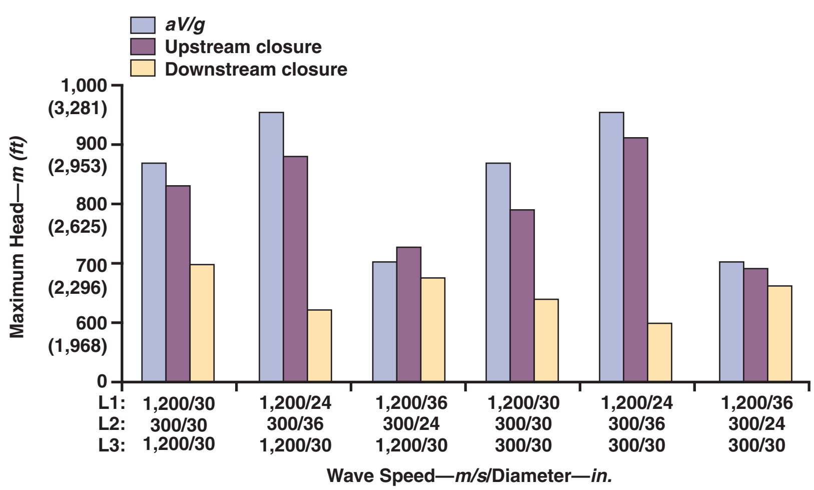
FIGURE 9 Transient response for downstream valve closure



a—*wave speed in m/s,* aV/g—*Joukowski surge head,* D—*diameter in in.*

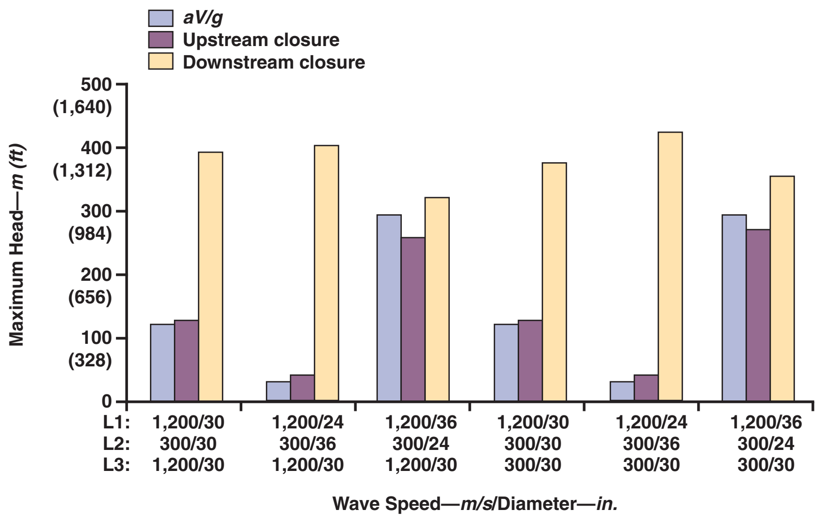
Properties are a_1/D_1 *(1,200/36),* a_2/D_2 *(300/24), and* a_3/D_3 *(300/30).*

FIGURE 10 Maximum head considering dead end



aV/g—*Joukowski surge head*

FIGURE 11 Minimum head considering dead end



aV/g—*Joukowski surge head*

closure. The upsurge attributable to the downstream valve closure is almost the same as the Joukowski surge pressure, but the minimum pressures are consistently ~40 m less. However, in this case overall agreement with the Joukowski relation is good. For a single pipeline subjected to a simple incident, the actual surge is well approximated by the potential surge.

Nonuniform pipeline. Another application encountered frequently in practice is a nonuniform series pipeline. In this study, the single pipeline consists of two pipes with the same length but with a stepwise change in diameter and/or wave speed. Table 2 shows wave speed, diameter, corresponding velocity, and Joukowski surge pressure in the two pipe sections. The first pipe has greater wave speeds, causing greater Joukowski surge pressure than anticipated in the second pipe; the smaller-diameter pipe also has the higher velocity, inducing a higher potential surge.

Figure 5 shows the transient response through a profile plot of the system for two pipes with the same diameter but different wave speeds; the first pipe has a wave speed of 1,200 m/s (3,940 fps), and the second pipe has a wave speed of 300 m/s (984 fps). The transient is initiated by closing the upstream valve. The first downsurge from the upstream valve is nearly the same as that predicted by the Joukowski relation. Yet when the wave arrives at the junction, a portion of the downsurge is transmitted downstream and some is reflected upstream. The figure clearly shows that the minimum head in the second pipe is now much lower than its Joukowski value because of the transmitted pressure but higher than the Joukowski downsurge of the first pipe. Similarly, Figure 5 shows that the reflected positive pressure in the first pipe is much lower than the Joukowski upsurge, because of the reflection at the junction, the friction along the pipeline, and the energy dissipation at the downstream reservoir.

Figure 6 depicts the maximum and minimum pressures caused by the downstream valve closure; the wave speeds are the same as in the Figure 5 case, but the diameters of the two pipes are now 36 in. (0.91 m) and 24 in. (0.61 m), respectively. The pressure envelope for each pipe section is again significantly different from the Joukowski analysis. The initial upsurge from the downstream valve is the same as in the Joukowski analysis, but the reflected wave from the junction increases the maximum pressure. When the initial upsurge is transmitted through the junction, the wave speed increases from 300 m/s (984 fps) to 1,200 m/s (3,940 fps), causing the increase in pressure head. However, if the pipe diameter increases from 24 in. (0.61 m) to 36 in. (0.91 m), there is a resulting decrease in head. Overall, the wave reflections complicate a Joukowski-based analysis.

Figures 7 and 8 summarize the maximum and minimum pressures for a sequence of runs in the two-pipe model characterized in Table 2 with the different location for the valve closure. Clearly the differences are much higher than those of Figures 3 and 4 because of the reflection at the junction. Another distinctly visible feature is that the location of valve closure affects the maximum and minimum pressure significantly, whereas the Joukowski analysis does not make this distinction. In the worst case, a head difference of ~400 m (1,310 ft) is shown in the minimum head (Figure 8). The overall conclusion drawn

> A more recently highlighted motivation for conducting a surge analysis arises from water quality considerations.

from this analysis is that the potential surge in complex systems is sometimes a conservative measure and at other times greatly underestimates the surge pressures.

Dead-end considerations. An issue often ignored but sometimes crucially important is the influence of dead ends on surge pressure. Do dead ends make simple rules and published guidelines more conservative or more dangerous? On the basis of a potential surge analysis, the dead end itself would not even be expected to experience a water hammer problem; moreover, dead ends are routinely purged from steady-state simulations because they have no direct hydraulic effect on system behavior. Why then should they be of concern in a water hammer analysis?

To test this case, the hypothetical system is similar to the nonuniform case described in the second study but with a dead end attached at the middle of the second pipe (1,200 m [3,940 ft]). The length, diameter, and Hazen-Williams friction factor of the dead end are 200 m (656 ft), 30 in. (0.762 m), and 120, respectively. Two wave speeds—1,200 m/s (3,940 fps) and 300 m/s (984 fps)—are selected to consider the influence of different pipe properties. The properties of pipe 1 and pipe 2 are the same as shown in Table 2, but the dead-end pipe (pipe 3) is analyzed using different wave speeds.

Figure 9 shows the transient response of the case in Figure 6 including a dead end with a wave speed of 300 m/s (984 fps). The dead end is located at 1,200 m (3,940 ft), causing the wave reflection from that location and making the system response more complicated than that shown in Figure 6.

Figures 10 and 11 show the maximum and minimum head for the system with the different locations of valve closure. As a comparison of Figures 10 and 11 with Figures 7 and 8 shows, the dead end influences the water hammer response in dramatic and important ways. Furthermore, its different wave speeds alter the system response, especially for the case in which a_1/D_1 = 1,200/36 and a_2/D_2 = 300/24. The reason the dead end can affect the maximum and minimum head significantly is that the surge pressure increase attributable to the wave speed increase conflicts with the surge pressure decrease attributable to the pipe diameter increase. Therefore the dead end located at pipe 2 affects the maximum and minimum head more significantly than do the other system conditions. In addition, Figures 10 and 11 indicate that the dead end causes surges considerably different from Joukowski predictions, depending on the system characteristics. One of the significant conclusions of these studies is that the rules of skeletonization and simplification that often remove dead ends in steady-state analysis or replace a multidiameter pipe with an "equivalent" one having similar head loss do not apply to transient applications.

Pipe network system. In most distribution systems, loops are formed to ensure system reliability and flexibility. The intention of this case study was to demonstrate how the use of the simplified Joukowski analysis could lead to incorrect conclusions for the transient response in a comparatively complicated network (looped) system.

The example pipe network is shown in Figure 12. The system comprises one reservoir at node 1, 45 pipes, and 29 nodes. This is a gravity-flow system that draws water from the reservoir to supply the network. The elevation of the reservoir at node 1 is 50 m (164 ft), and all

FIGURE 12 Pipe network

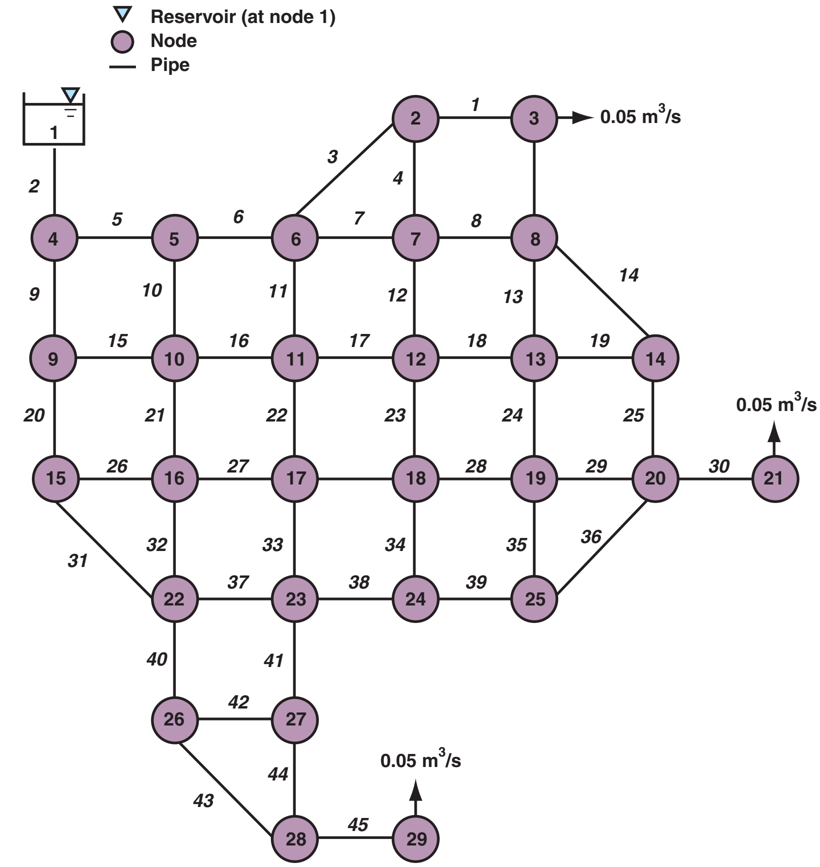
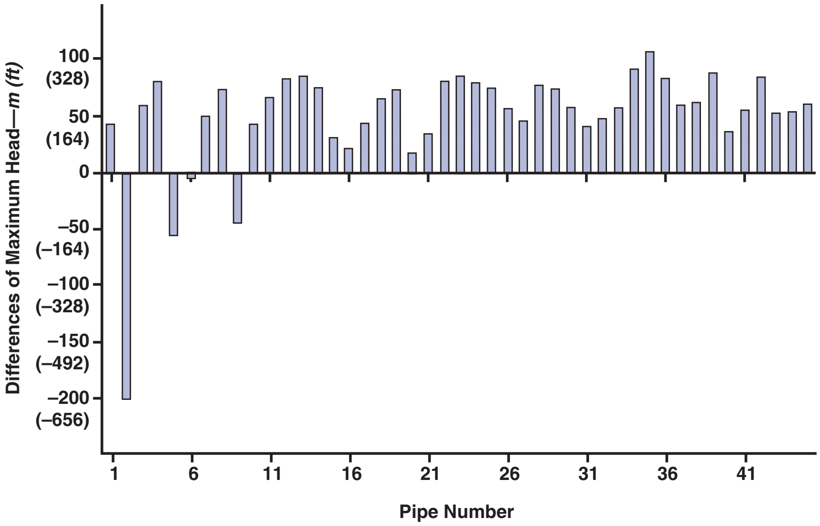


FIGURE 13 Difference between the maximum heads and Joukowski upsurge



other nodes have zero elevation. For simplicity, the length, diameter, wave speed, and Darcy-Weisbach friction factor of all pipes are 500 m (1,640 ft), 0.3 m (0.984 ft), 1,200 m/s (3,940 fps) and 0.015, respectively. At the end of the network, three 50-L/s (1.77-cu ft/s) demands at nodes 3, 21, and 29 are considered here. In order to introduce transient conditions into the case study, the valves at nodes 3, 21, and 29 are closed instantaneously to introduce a rapid transient into the system. Although clearly an arbitrary and somewhat dramatic transient load, the difficult question any analyst faces in practice is this: "What loading cases are appropriate and suitably severe?" Historically, little thought or reflection has been given to this important question.

Figure 13 shows the difference on the maximum heads between a detailed surge analysis and a simplified Joukowski one. Results clearly demonstrate that the Joukowski analysis results are not suitable to estimate the transient response in most pipes. In the worst case, the difference in surge pressure predictions for pipe 2 is more than 200 m (656 ft). This is because its steady-state velocity is higher than the other pipes, which causes the greater Joukowski upsurge. Another interesting and important feature of the results is that the system responses determined using the detailed surge analysis are worse than the Joukowski upsurge computed on the basis of initial pipe velocities for pipes in the middle of the network. The Joukowski upsurge and downsurge may be more severe (and thus conservative) than results from more-detailed surge analysis in some systems; however, the opposite effect unfortunately is not rare in looped networks. In addition, the inability of the Joukowski rule to predict reasonable surge pressures is apparent if the wave speeds in pipes 1, 30, and 45 are changed to 300 m/s (984 fps) instead of 1,200 m/s (3,940 fps). These pipes are located next to the valves at nodes 3, 21, and 29, so the decreases of the wave speed eventually affect the system response in all the pipes. However, Joukowski analysis decreases the surge pressure prediction only in pipes 1, 30, and 45.

Another noticeable defect of the Joukowski rule is its inability to simulate a variety of loadings in the quest for the worst-case scenarios in a distribution network system. Logic using the Joukowski relation ignores wave reflections at the different pipe properties and the probability of conjunctive events in a distribution system, which can significantly magnify or attenuate the water hammer wave. Moreover, the Joukowski rule cannot consider liquid column separation in a pipeline. The pressure below the vapor pressure of a liquid may produce vapor cavities in the flow, and the collapse of the cavities results in a large pressure rise, which may damage the pipeline system. Its occurrence may have a significant effect on subsequent transients in the system. Therefore, the simplified surge analysis cannot provide a reliable tool for estimating the risk of water hammer.

Of the many challenges that face water utilities, one critical but too-often-forgotten issue is protecting the system from excessive transient or water hammer conditions.

CONCLUSIONS

Water hammer analysis is important, but its complexity (perhaps coupled with its mysterious nature and the need for specialized analysis tools) has led to a number of published guidelines promoting simplifications in the analysis. AWWA literature in particular suggests simplified rules to estimate water hammer phenomena; this study showed these rules to be inaccurate in certain cases and thus likely to lead to poor designs.

Most simple expressions, such as the Joukowski relation, are applicable only under a set of highly restricted and often unrealistic circumstances. When the required conditions are met, the simple relationships are both powerful and accurate. In the case of the Joukowski relation, the two most important restrictions are that there should be only a small head loss resulting from friction and no wave reflections from any hydraulic devices or boundary conditions in the system. If these conditions are not met, the Joukowski expression is no longer valid and the conclusions based on this rule also may not be applicable. Moreover, the Joukowski relation does not consider liquid column separation. If a negative surge is below the vapor pressure, all gas within the water is gradually released, and the collapse of the cavities will result in a large pressure surge spike. Of course, the question might be raised as to whether system designers or water utilities can afford to complete a transient analysis. To this question, the authors pose another: Given the importance of this analysis and the magnitude of the errors that overly simplified rules can lead to, can utilities afford not to be comprehensive in their analyses of their distribution systems?

No simplified rules can provide a prediction of the worst-case performance under all transient conditions. The water hammer response in distribution systems is strongly sensitive to system-specific characteristics, and any careless generalization and simplification could easily lead to incorrect results and inadequate surge protection. Comprehensive water hammer analysis is not only needed, but this approach is both justified by its importance and practical, thanks to the rapid development of fast computers and both powerful and efficient numerical simulation models for water hammer analysis.

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UNI-BELL TECHNICAL REPORT

SUSTAINABLE PIPE INFRASTRUCTURE –
IT'S OUR RESPONSIBILITY

PVC FORCE MAIN DESIGN

TABLE OF CONTENTS

Introduction

Due to the high cost of wastewater collection systems, engineers are continually re-evaluating old ideas concerning wastewater collection. Gravity-flow collection systems normally offer the most practical and cost-effective solution to a community's collection needs. However, the gravity flow solution is sometimes economically impractical or physically impossible. To overcome these problems, engineers may elect to install pump stations and force mains.

A force main is defined as a piping system that transports sanitary sewage with internal pressure. High pressure is created at the system's intake point by a pump. This pump station "forces" the wastewater to the discharge, or low-pressure point, in the system.

While a wealth of information exists on pump-station design, examples and design guidelines for sewer force mains are often inadequate. Uni-Bell has chosen to produce a design manual which deals directly with the issues surrounding a force main sewer installation using PVC. Although the following design manual will briefly touch on pump design requirements, pump-station design is not included.

Material Selection

Upon selecting PVC pressure pipe for force main design, the designer forms a solid foundation for the short- and long-term success of the project. Polyvinyl Chloride (PVC) piping material has withstood the test of time. Extensive performance records have been compiled proving PVC is the piping material of choice for sanitary sewer applications.

There are numerous reasons for PVC's success in sanitary sewer applications, the first and foremost being PVC's ability to resist corrosion. PVC's inert nature is attributed to the fact that vinyl is a non-conductor of electricity, making it immune to electrochemical reactions caused by acids, bases, and salts. This inert characteristic is very important for sanitary sewer installations, where aggressive environments exist both outside and inside the system.

PVC's ability to resist corrosion is an advantage not shared by traditional pressure-piping materials. By specifying PVC pipe for force-main design, the designer has chosen to eliminate the possibility of corrosion-induced pipe failure.

PVC pipe also offers other distinct advantages. An immediate benefit will be construction cost savings through PVC's light weight and ease of installation. Appreciable financial savings can be realized during any project's construction phase by eliminating the heavier equipment needed to install traditional piping materials. In addition to initial savings, PVC's superior hydraulic characteristics will often result in lower lifetime costs for pumping and maintenance of the system.

The major total costs of a pumping system include the cost of pumps, pipes, installation, operation and maintenance (O&M), and energy. A larger diameter pipeline (higher initial cost) will result in lower friction head loss and require a smaller total pumping head with lower horsepower pumps and less energy (lower lifetime cost). A PVC pipeline offers the same lower lifetime costs without the expenditure of high initial material costs by providing competitive initial costs, long service life, low maintenance costs, and lower friction head loss achieved through its hydraulic smoothness.

Hydraulic Design

Hydraulic research and analysis have shown that flow conditions in PVC pressure piping systems can be designed conservatively using the Hazen-Williams equation. Flow conditions may also be designed with more detailed analysis using the Darcy-Weisbach equation.

The Hazen-Williams flow formula is most widely accepted and used in the calculation of pressure pipe flow conditions.

EQUATION 1

$$V = 1.318\,C\left(R_H\right)^{0.63}\left(S\right)^{0.54}$$

Where:

- **V** = flow velocity, ft/s
- **C** = flow coefficient
- **R_H** = hydraulic radius, ft
  Note: The hydraulic radius is defined as the flow area divided by the wetted perimeter in the interior of the pipe.
  Note: R_H = 1/4(D_i) for pipe flowing full or half full
- **S** = hydraulic slope, ft/ft
- **D_i** = pipe inside diameter, ft

Research has established that the Hazen-Williams flow coefficient, or “C” factor, is commonly defined in a range of values from 155 to 165 for both new and used PVC pipe. The Hazen-Williams “C” factor, therefore, has been established conservatively at C = 150 for the design of PVC piping systems.

Pipe sizing and pump design and selection are topics covered in many civil engineering textbooks. The intent of this document is not to repeat this material. Rather, the intent is to give details on the specifics of PVC pressure-pipe design and installation, as well as highlighting design considerations that the textbooks sometimes lack.

Some of the basic details the designer gathered when optimizing the pipe and pump details are listed below:

- Pipe material (PVC)
- Projected wastewater flows
- Pipe profile and stationing

Once the designer has determined the basic hydraulic components for the project, the pump curve and system curve are used to determine the pressures and flows within the system for various operating conditions. The designer then establishes the hydraulic profile for the project. At that point, the designer has the information needed to specify the appropriate class of PVC for the pressure requirements of the project.

PVC Pressure Pipe Design

Consistent with all thermoplastic materials, PVC has three distinct strength limits. These are: (a) Long-Term Strength, (b) Short-Term Strength, and (c) Cyclic Strength. The sewage force main designer must select a PVC pipe that is within the limits of all three strength categories.

The design approach presented in this manual will include three separate design checks, listed below:

- The system's operating pressure will establish the required long-term pressure rating (PR) of the PVC pipe, based on a Factor of Safety of 2.0 against the Hydrostatic Design Basis.

- The worst-case, non-recurring surge experience will establish the required short-term pressure rating (STR) of the appropriate PVC pipe, based on a Factor of Safety of 2.0 against the quick-burst strength.

- A cyclic fatigue check, using the design tools developed through research conducted by Utah State University, will establish a fatigue life that meets or exceeds the desired design life.

<u>Long-Term Strength</u>

This is the design step that is familiar to most engineers. The operating pressure is compared to the PR of the pipe. The PRs for various dimension ratios of PVC pipe are listed in Table 1. (With the latest revision of the AWWA C900 standard, the Pressure Class (PC) is now the same as the Pressure Rating (PR) for a given dimension ratio.) Mathematically, the Standard Dimension Ratio (SDR) is the same as the Dimension Ratio (DR). Both are defined in Equation 2.

TABLE 1

Pressure Ratings for PVC Pipe

SDR or DR	PR or PC, psi (MPa)
51	80 (0.55)
41	100 (0.69)
32.5	125 (0.86)
26	160 (1.10)
25	165 (1.14)
21	200 (1.38)
18	235 (1.62)
14	305 (2.11)

EQUATION 2

$$\mathbf{SDR = DR = D_o \div t_{min}}$$

Where: D_o = average outside diameter
t_{min} = minimum wall thickness, same units as D_o

The PR selected must exceed the normal operating pressure determined by the hydraulic design of the system.

Short-Term Strength

An inherent property that PVC pipes have always offered is higher pressure capacity as the duration of the applied pressure is decreased. This is why the quick-burst and sustained-pressure test values used for quality-control purposes far exceed a PVC pipe's Pressure Rating. An SDR41 product, with a 100 psi Pressure Rating, serves as an example. For the sustained-pressure test, an SDR41 must hold 210 psi in excess of 1,000 hours. For the quick-burst test, the pressure causing the pipe to burst must exceed 315 psi, where the time to burst is calibrated to be in the 60 to 70 second range.

TABLE 2
STR Values for PVC Pipe

SDR or DR	STR, psi (MPa)
51	128 (0.88)
41	160 (1.10)
32.5	203 (1.40)
26	256 (1.77)
25	264 (1.84)
21	320 (2.21)
18	376 (2.60)
14	488 (3.40)

For non-recurring pressure surges, such as those resulting from a power outage, select an STR that exceeds the peak pressure (including surge).

If better data are not available from transient analysis software, a conservative estimate for a non-recurring pressure surge may be calculated using Equation 3 and Table 3. This assumes that the surge resulted from an operation that instantaneously stopped the column of wastewater.

EQUATION 3

$$P_{peak} = P_{op} + V(P_s')$$

Where:
- P_{peak} = peak pressure from non-recurring surge event, psi
- P_{op} = normal operating pressure, psi
- V = maximum flow velocity, ft/s
- P_s' = surge for a 1 ft/s velocity change (Table 3), psi

Table 3
Pressure Surge versus Dimension Ratio
(In Response to a 1 ft/s (0.3 m/s) Instantaneous Flow Velocity Change)

SDR or DR	Pressure Surge, P_S' psi (kPa)
51	10.8 (74)
41	11.4 (79)
32.5	12.8 (88)
26	14.4 (99)
25	14.7 (101)
21	16.0 (110)
18	17.4 (120)
14	19.8 (137)

Cyclic Strength

The Utah State University (USU) research on the cyclic capabilities of PVC pressure pipe [Jeffrey, 2004] confirmed that predicting the fatigue capabilities of PVC is much like that of any other material. It is a function of two variables: the average stress and the stress amplitude. (See Equations 4 and 5.) The design chart developed by the USU research is shown in Figure 1.

EQUATION 4

$$\sigma_{avg} = \frac{(P_{max} + P_{min})(DR - 1)}{4}$$

Where:
- σ_{avg} = required average hoop stress, psi
- P_{max} = maximum recurring pressure, psi
- P_{min} = minimum recurring pressure, psi
- **DR** = dimension ratio of the pipe

EQUATION 5

$$\sigma_{amp} = \frac{(P_{max} - P_{min})(DR - 1)}{4}$$

Where:
- σ_{amp} = required design stress amplitude, psi

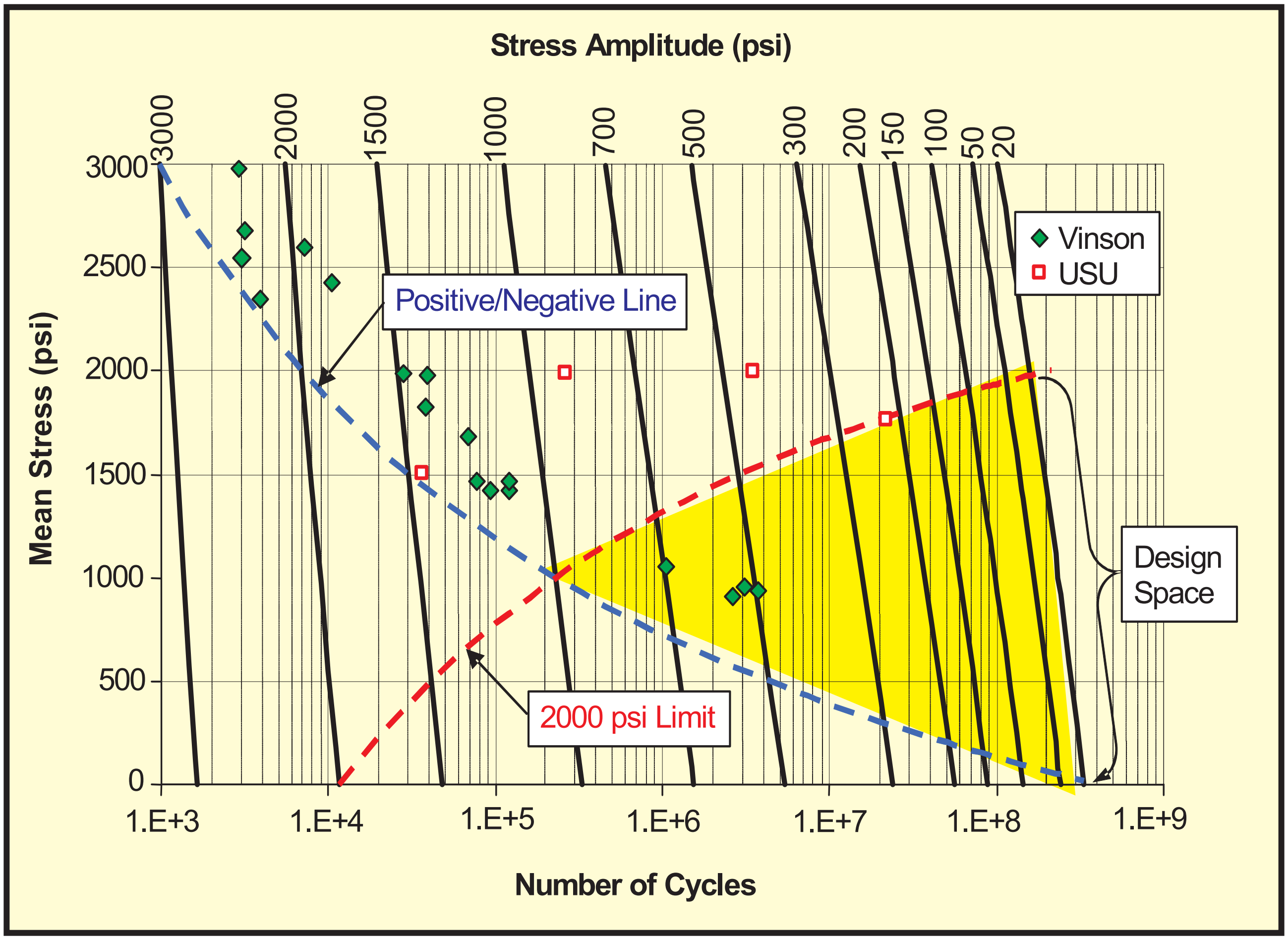


Figure 1: Design Chart for the Cyclic Strength of PVC Pressure Pipe

The successful design will result in a cyclic life (**C**) that exceeds the total number of cycles (**C′**) that occur throughout the design life. Figure 1 is used to determine **C**, while Equation 6 is used to calculate **C′**. Note that Equation 6 uses the total number of cycles per day, not the number of cycles per hour. By using a total daily number, both heavy and light periods of sewage flows are accounted for. The worst case design condition would be based on the design number of starts per hour, which is part of the pump selection and wet well design.

EQUATION 6

C′ = (N cycles/day) (365 days/year) (Design Life in Years)

PVC Pressure Pipe Design Example

This design example will demonstrate the various design checks for PVC pressure pipe. The pipe under consideration is a 14-inch SDR41 AWWA C905 pipe, with a CIOD diameter regimen. The design flow is 3.03 cfs. It is estimated that the pump will have 58 starts and 58 stops each day. Once the steady-state-operating flow has been achieved, the operating pressure is 27 psi. During start-up or shut-down, the pressure amplitude may be as large as 20 psi. The force main discharges into a manhole at atmospheric pressure at the upstream end. The desired design life is 50 years. An analysis of transients using design software has not been conducted.

EQUATION 7

$$D_i = D_o - 2t'$$

Where:
- D_i = inside diameter, in
- D_o = outside diameter, in
- t' = $t_{min} + 0.5\, t_{tol}$, in
- t_{min} = minimum wall thickness, in
- t_{tol} = tolerance on the minimum wall thickness, in

EQUATION 8

$$A_x = \frac{(\pi \div 4)(D_i)^2}{144}$$

Where:
- A_x = flow cross sectional area, ft^2

EQUATION 9

$$V = Q \div A_x$$

Where:
- V = flow velocity, ft/s
- Q = flow, ft^3/s
- A_x = flow cross sectional area, ft^2

The following information may be found in Table 2 of the AWWA C905 standard.

D_o = 15.300 in

t_{min} = 0.373 in
t_{tol} = +0.052 in

With that information and Equation 7, the average inside diameter may be computed.

t' = 0.373 + (0.5) (0.052) = 0.399 in
D_i = 15.300 - 2 (0.399) = 14.502 in

Next, find the flow cross sectional area using Equation 8 and the fluid velocity using Equation 9.

A_x = $[(\pi \div 4)(14.502)^2] / 144$ = 1.147 ft^2
V = (3.03) ÷ (1.147) = 2.64 ft/s

Routine pressures the force main experiences are as follows:

P_{op} = Operating pressure = 27 psi (Known)
P_{amp} = Pressure amplitude = 20 psi (Known)
P_{max} = 27 + 20 = 47 psi
P_{min} = 27 - 20 = 7 psi

Hoop stresses from the routine pressures may now be calculated using Equations 4 and 5.

σ_{avg} = [(47 + 7) (41 - 1)] ÷ 4 = 540 psi
σ_{amp} = [(47 - 7) (41 - 1)] ÷ 4 = 400 psi

The number of cycles per day (**N**) may be determined from the known information. Equation 6 may be used to determine the number of cycles that occur throughout the design life.

N = 58 starts + 58 stops = 116 cycles / day
C' = (116) (365) (50) = 2.12×10^6 cycles

This system experiences 42,340 cycles per year.

Check Long-Term Strength

Is the operating pressure less than or equal to the PR of the SDR41 selected?

$P_{op} \le PR$?
27 psi ≤ 100 psi (From Table 1)
Yes. The first design check is passed.

Check Short-Term Strength

Equation 3 will be used to estimate the worst-case, non-recurring pressure. Table 3 lists the $\mathbf{P_s'}$ for SDR41 as 11.4 psi.

$$\mathbf{P_{peak}} = 27 + (2.64) (11.4) \qquad = 57 \text{ psi}$$

Now the adequacy of the SDR41's short-term strength may be checked.

Is the peak pressure less than or equal to the STR of the SDR41 selected?

$P_{peak} \leq$ STR?
58.7 psi ≤ 160 psi (From Table 2)
Yes. The second design check is passed.

Check Cyclic Strength

Is the cyclic capacity greater than the anticipated number of cycles expected for the force main over its design life?

$C' \leq C$?
$2.12 \times 10^6 \leq 9 \times 10^6$ (From Figure 1)
Yes. The third and last design check is passed.
The design is satisfactory.

Cyclic Check, Step-by-Step

The manner by which the 9.0×10^6 value for **C** was determined is now further detailed. Refer to Figure 2. First note that it is a semi-log chart. The x-axis has a logarithmic scale. One of the two independent variables, average stress, serves as the chart's y-axis. The second independent variable, stress amplitude, is represented by the black diagonal lines that overlay the chart. Each line represents a different stress amplitude. As one moves from left to right across the chart, the stress amplitudes represented by the black lines decrease. The x-axis is the dependent variable, and it denotes the cyclic life (**C**) of the PVC pressure pipe.

In this example, the first independent variable, σ_{avg}, has a value of 540 psi. In Figure 2, that value is shown in green on the y-axis. A green line extends right from that value until it intersects the second independent variable, σ_{amp}. In the example, σ_{amp} has a value of 400 psi. However, the design chart does not have a line for a 400 psi σ_{amp}. There are σ_{amp} lines for 300 and 500 psi. Since 400 falls halfway between those two values, a purple line was added to the chart midway between those values to represent the 400 psi σ_{amp} line. At the intersection of the green line and the purple line, the tail of a blue line is shown. The blue line is extended downward until it hits the x-axis. The value for **C** in this example is 9.0 x

10^6, which is read from the x-axis of the chart at the point of intersection of the arrowhead of the blue line and the x-axis.

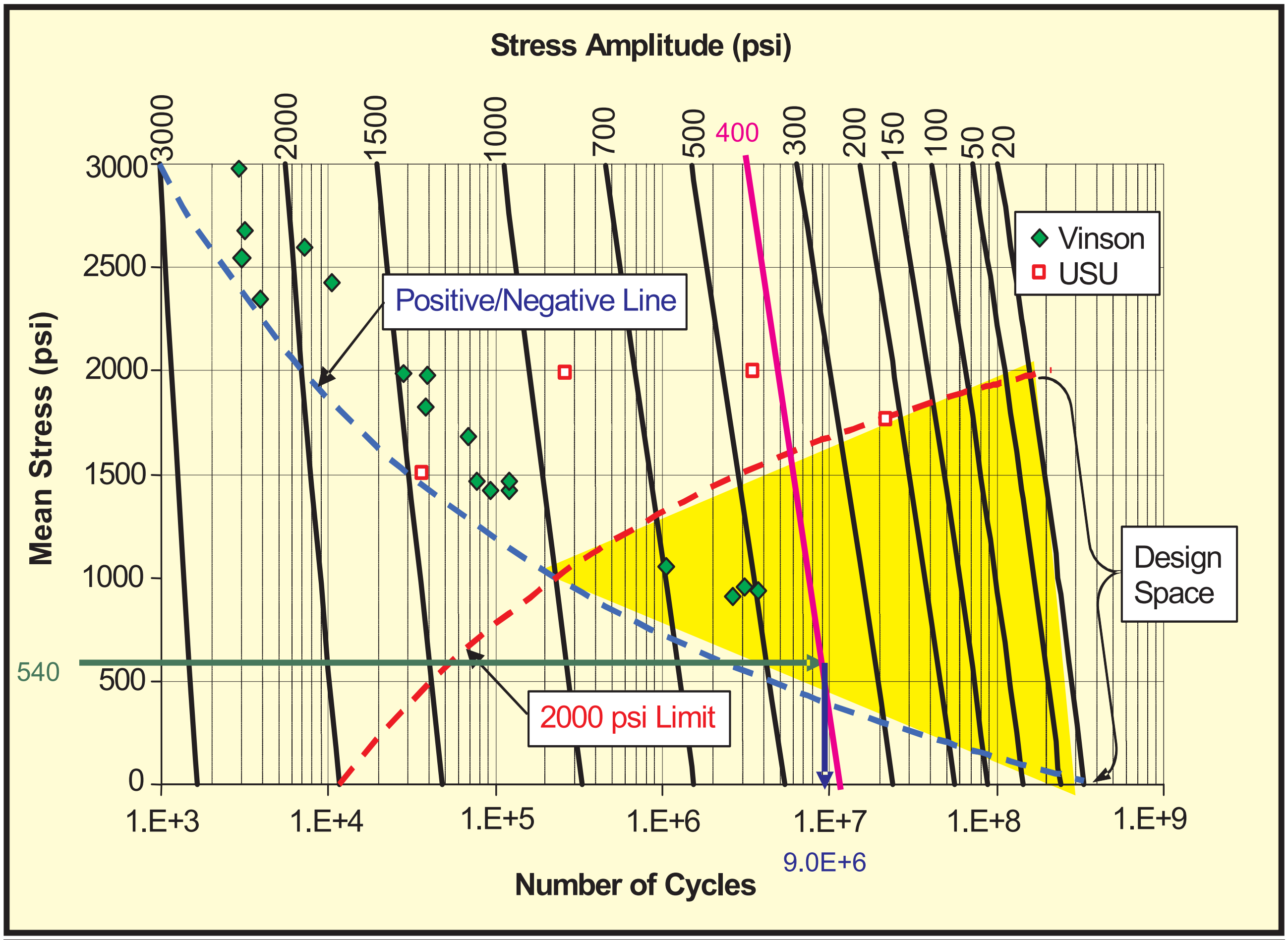


Figure 2: Using the Design Chart for the Design Example

<u>Cyclic Life</u>

At 42,340 cycles per year, what is the cyclic life for the pipe in this design example?

Cyclic Life = **C** ÷ 42,340 cycles / year
= 9.0×10^6 ÷ 42,340 cycles / year
= 213 years

Note: Large pressure swings have been greatly reduced in recent years with the use of “soft-start / soft-stop” pumps in force main applications. The pressure swing shown in this example could be further reduced by taking full advantage of this technology or other surge control devices. There have also been improvements in variable speed pumping, which has increased its popularity. If variable-speed pumping were employed, the number of starts and stops shown in this example would be greatly reduced as would the pressure swings.

Additional Design Considerations

<u>Common Surge-Control Techniques</u>

Pressure-control devices serve multiple tasks within a force main. Their primary function is to minimize pressure fluctuations created when a change in fluid velocity occurs within the system. A change in flow velocity within a closed conduit causes elastic waves to travel upstream and downstream from the point of origin. These elastic waves cause an increase or decrease in pressure as they travel along the line. These pressure changes are referred to as water hammer, surge pressure, or transient pressure.

Where either the magnitude or frequency of surges is judged to be the limiting parameter in a pipeline design, there are practical means of reducing them to acceptable levels. In general, the first objective is to keep the upsurge and downsurge (the maximum positive and negative surges) at minimum values. Within this minimized transient pressure envelope, even at a fixed-cycle frequency (i.e., where **C** is not a controllable variable), the operation of the pipeline may often proceed because the cyclic strength of the system is shown to be sufficient. Due to the wide variety of surge conditions possible, positive or negative pressures, transient or oscillatory, there is no general solution applicable to the control of surge conditions.

There are various means of controlling, reducing, or withstanding surge pressure in pressure systems. One method is to use variable speed pumps, which allow the pumping operation to be continuous for varying flow conditions, thereby greatly reducing the amount of on/off cycles. Other options are sketched in Figure 3 and are described below:

- **Controlled Closing Check or Pump Control Valve:** Slowly opening and closing check valves or pump control valves are an effective way of controlling pressure surges during normal pump starts and stops. (This arrangement is often referred to as a "soft start / soft stop" pump.) The rate of opening and closing will be a function of the pipe length. Another factor in the design is the minimum flow required by volute-casing-centrifugal pumps. The pump manufacturer should be consulted for the flow required to relieve the high radial thrusts this type of pump generates. (See Figure 3a.)
- **Air Exhaust and Air Inlet Valves:** The air exhaust valves will serve two purposes. The first will be to vent air at the high points when the

line is slowly filled for its acceptance testing after installation. The second purpose will be to vent the entrained air that comes out of the wastewater between pump cycles. Air inlet valves will admit air when the pressure drops below atmospheric pressure in order to prevent a vacuum from occurring. (See Figure 3b.) This is discussed further in the next section.

- **Pressure Surge Relief Valves:** Spring-loaded valves which release and vent pressures in excess of a pre-set value. When activated, the wastewater discharged is piped back to the wet well. (See Figure 3c.)
- **Closed or Pressurized Surge Tanks:** A closed unit containing air and wastewater occasionally separated by a diaphragm or a bladder. The air is under pressure allowing control of both positive and negative surges in high-pressure systems by allowing flow into and out of the unit. (See Figure 3d.)
- **Surge Tower:** A tank open to the atmosphere that functions in a manner similar to a surge tank for low pressures. (See Figure 3e.)
- **Pump and Driver Inertia:** Pumps which decelerate slowly in the event of a power outage, which minimizes the downsurge on the wastewater column. (See Figure 3f.)

Proper maintenance of the surge control devices and other system appurtenances is necessary for the system to continue to operate as designed.

PVC force mains can utilize some or all of the methods listed above, and Figure 3 illustrates these approaches. Additional discussion is provided in “Design of Wastewater and Stormwater Pumping Stations,” [WEF MOP FD-4, 1993].

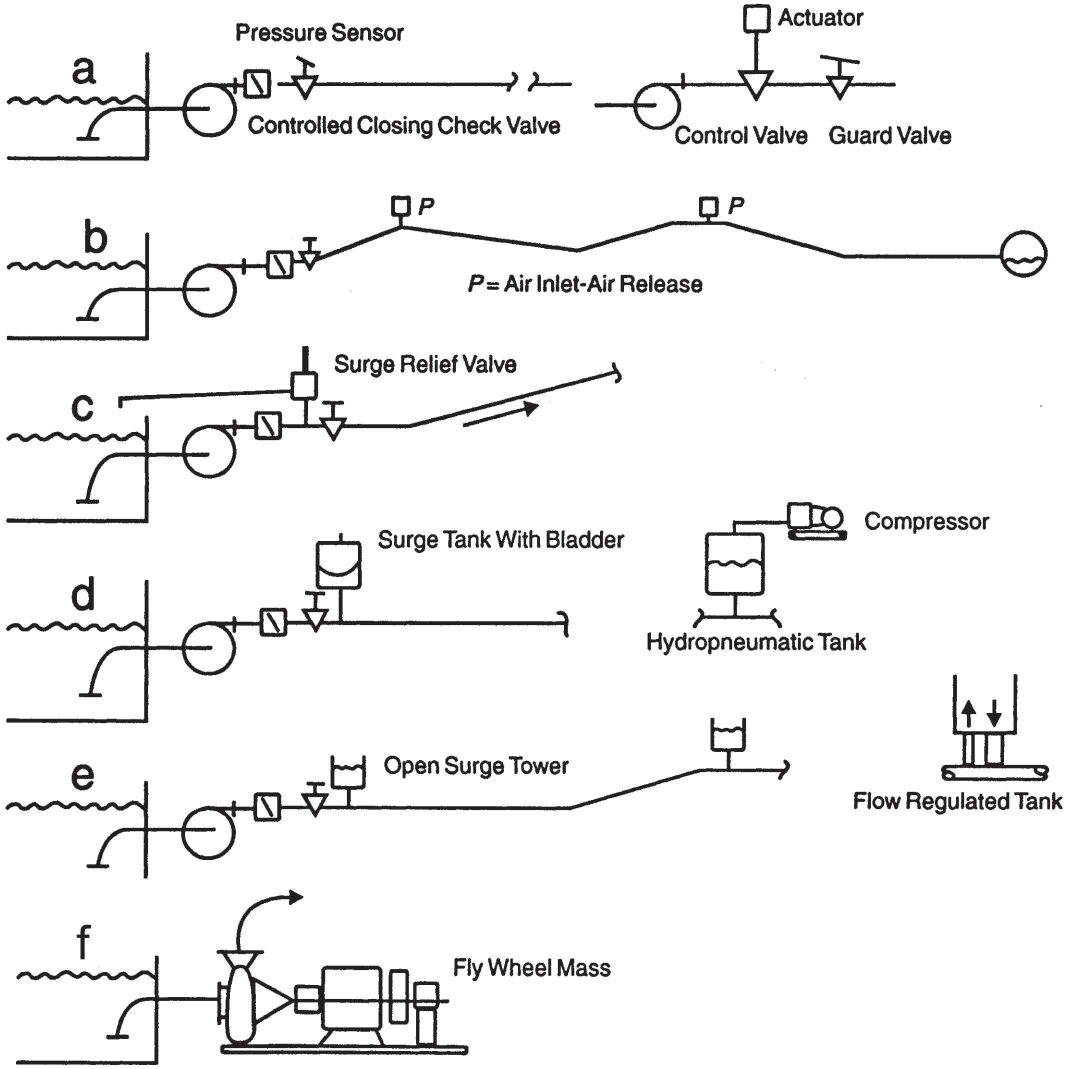
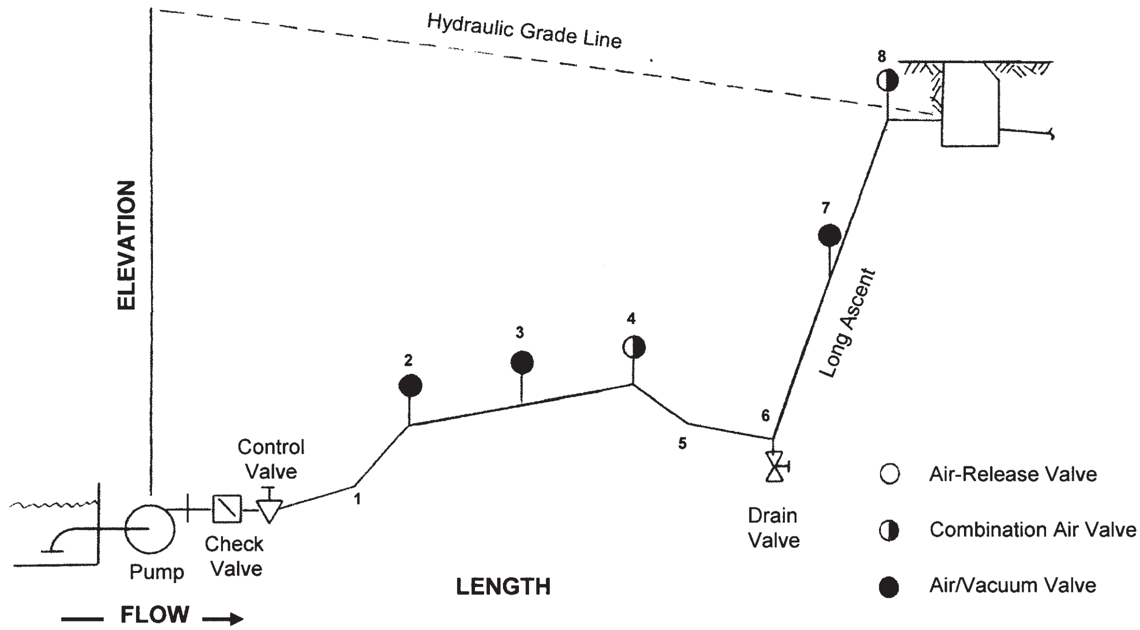


Figure 3: Surge Reduction Methods [WEF MOP FD-4, 1993]

<u>Entrapped Air</u>

Air in piping systems will tend to collect at high points in the line when flow velocities are low. "Recommended Standards for Sewage Works" [GLUMRB, 2004] states that automatic air-release valves should be placed at high points along the force main to prevent the accumulation of entrapped air. "Air-Release, Air/Vacuum, and Combination Air Valves" [AWWA M51, 2001] has recommendations for the type of valve and where it should be located. The applicable recommendations are:

- **High Points:** Combination air valves should be installed at pipeline high points to provide venting while the force main is filling, during normal operation, and for air inflow and vacuum protection while the pipe is draining.
- **Increased Downslope:** A combination air valve should be considered at abrupt increases in downslope.
- **Decreased Upslope:** An air/vacuum valve or a combination air valve should be considered at abrupt decreases in upslope.
- **Long Ascents and Long Descents:** An air/vacuum valve or combination air valve should be considered at intervals of a quarter mile to a half mile along ascending sections of pipelines.
- **Horizontal Runs:** Combination air valves should be considered at the beginning and end of long horizontal sections, and air-release valves or combination air valves should be considered at intervals of a quarter mile to a half mile along horizontal sections of pipelines. It is difficult to evacuate air from a long horizontal pipeline at low-flow velocities.



Point	Description	Type	Point	Description	Type
1	Increasing Upslope	No Valve Required	5	Decreasing Downslope	No Valve Required
2	Decreasing Upslope	Air/Vac or Combination	6	Low Point	No Valve Required
3	Long Ascent	Air/Vac or Combination	7	Long Ascent	Air/Vac or Combination
4	High Point	Combination	8	Decreasing Upslope	Air/Vac or Combination

Figure 4: Locating Air and Vacuum Valves on a Typical Force Main

Refer to AWWA M51 (2001) for the design approach for sizing the valve or consult the valve manufacturer. Proper sizing of air valves and other surge control devices can also be obtained by using transient-analysis computer software. Figure 4 shows a typical force main and locations of air valves. Consultation with the valve manufacturer is recommended whenever the force main projects above the hydraulic grade line.

Reflected Pressure Waves

In the PVC pressure-pipe design example, no secondary or tertiary pressure waves are generated during a pump start-up or shut-down because the system is vented to the atmosphere. If the system were a closed system, and if surge pressures were able to be reflected, additional analysis using Miner's Rule would be needed to determine the system's cyclic life. (See Appendix A.)

The cyclic life is now recalculated with the following assumptions:

- The system is closed and the surge pressure wave is reflected throughout the pipeline.
- The attenuation of the secondary and tertiary waves follows a dampened sinusoidal pattern as shown in Figure 5.

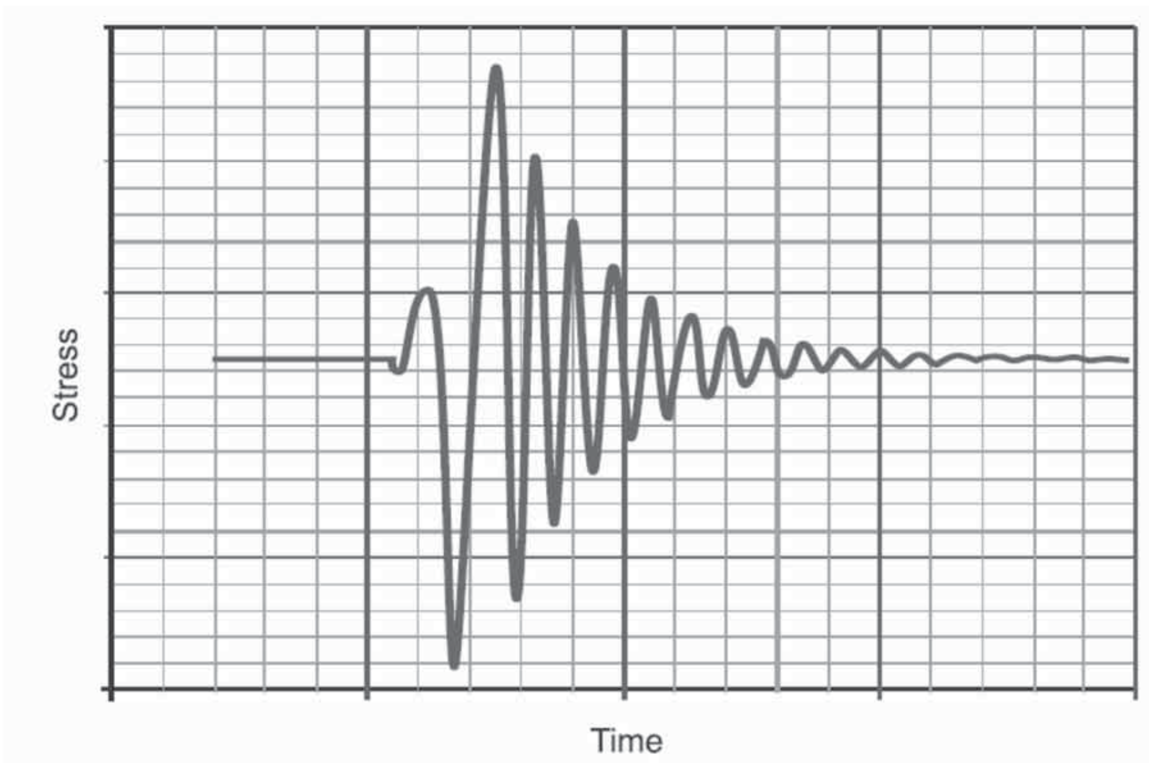


Figure 5: Typical Surge Pattern
Pressure Decay is Represented by a Dampened Sinusoidal Pattern

The first step is to represent the pressure pattern with the equivalent number of primary pressure waves. Jeffrey (2004) shows that the secondary and tertiary waves have the fatigue equivalent of 0.55 primary waves. So, the number of cycles per year must be increased by 55% to account for the pressure pattern generated by reflected surge pressure waves.

$$\text{Primary Cycles per Year} = (1.55)\ (42{,}340)$$
$$\text{Primary Cycles per Year} = 65{,}600 \text{ cycles/year}$$

$$\text{Cyclic Life} = C \div \text{Anticipated Number of Primary Cycles per Year}$$

$$\text{Cyclic Life} = 9.0 \times 10^6 \div 65{,}600 \text{ cycles/year}$$
$$\text{Cyclic Life} = 137 \text{ years}$$

The design is again shown to be more than satisfactory from a cyclic fatigue point-of-view. Appendix A and the article by Fisher (2004) show more of the intermediary steps used for deriving the 55% value given by Jeffrey (2004).

External Load Design

The external load design requirements for PVC force main design do not vary from what is required of any PVC pressure piping system. Methods of external load design have been extensively covered and Uni-Bell offers the documents referenced below as recommended external load design practices for force main installations:

- **PVC Pipe - Design and Installation**, American Water Works Association Manual of Water Supply Practices M23, AWWA, Denver, CO, 2002.

- **Handbook of PVC Pipe: Design and Construction**, Fourth Edition, Uni-Bell PVC Pipe Association, Dallas, TX, August, 2001.

- **Deflection: The Pipe/Soil Mechanism**, **UNI-TR-1-97**, Uni-Bell PVC Pipe Association, Dallas, TX, April, 1997.

- **External Load Design for Flexible Conduits**, design software available for download from Uni-Bell's website, www.uni-bell.org.

Installation Procedures

Proper installation procedures are critical for ensuring the longevity of the sewer force main. Research has established best practices for the installation of PVC potable water distribution and transmission pipelines. Uni-Bell offers the guidelines referenced below as recommended practices for the installation of PVC force-main pipe:

- **PVC Pipe - Design and Installation**, American Water Works Association Manual of Water Supply Practices M23, AWWA, Denver, CO, 2002.

- **Underground Installation of PVC Pressure Pipe and Fittings for Water**, American Water Works Association Standard C605-05, Denver, CO, June, 2005.

- **Handbook of PVC Pipe Design and Construction**, Fourth Edition, Uni-Bell PVC Pipe Association, Dallas, TX, August, 2001.

Acceptance Testing

To ensure the system has been installed properly, the installer should perform hydrostatic testing of the completed system. Deflection testing is not typically performed on PVC force sewer main installations. Hydrostatic test procedures have been established previously for clean water distribution and transmission and are applicable for acceptance testing of force main installations. Uni-Bell offers the guidelines referenced below as recommended practices for the acceptance testing of PVC force main installations:

- **PVC Pipe - Design and Installation**, American Water Works Association Manual of Water Supply Practices M23, AWWA, Denver, CO, 2002.

- **Underground Installation of PVC Pressure Pipe and Fittings for Water**, American Water Works Association Standard C605-05, Denver, CO, June, 2005.

- **Handbook of PVC Pipe Design and Construction**, Fourth Edition, Uni-Bell PVC Pipe Association, Dallas, TX, August, 2001.

The prudent engineer may wish to confirm the hydraulic design by recording actual pressures with a pressure transducer data-logger after the line is put in service. To accommodate the installation of a pressure transducer, a fitting with the appropriate threaded port should be installed downstream of the pump and just past the check valve. In the event that unusual pressure surges are recorded, remedial transient analysis may be done to identify the appropriate surge controls to rectify the situation.

Additional References

"Air-Release, Air/Vacuum, and Combination Air Valves, Manual of Water Supply Practices M51, American Water Works Association, Denver, CO, 2001.

"Design of Wastewater and Stormwater Pumping Stations," Manual of Practice FD-4, Water Environment Federation, Alexandria, VA, 1993.

Fisher, Craig, “The Anatomy of a Force Main Pressure Wave,” PVC Pipe News, Uni-Bell PVC Pipe Association, Dallas, TX, Summer, 2004.

Jeffrey et al, “Long-Term Cyclic Testing of PVC Pipe,” Utah State University, Buried Structures Lab, Logan, UT, February 26, 2004.

“Recommended Standards for Wastewater Facilities,” Great Lakes-Upper Mississippi River Board of State and Provincial Health and Environmental Managers, Health Education Services, Albany, NY, 2004.

APPENDIX A

Cyclic Cumulative Damage

The following discussion was adopted from portions of Appendix A of Jeffrey (2004).

Cyclic Cumulative Damage

For cyclic pressures where the amplitude is variable, a method for determining the influence that each amplitude has on the total cyclic life is necessary. The linear-cumulative-damage rule, or Miner's Rule, has come into common use following its publication by M. A. Miner in 1945. The linear-cumulative-damage rule assumes that the total life can be estimated by adding up the percentage of life consumed by each stress cycle.

Miner's Rule is expressed by Equation A1 [Juvinall, 1967], in which $\mathbf{n_1, n_2, \ldots, n_k}$ represent the number of cycles at specific stress levels and $\mathbf{N_1, N_2, \ldots, N_k}$ represent the life (in cycles) at these stress levels, as taken from the S-N curve (Figure 1).

EQUATION A1

$$\frac{n_1}{N_1} + \frac{n_2}{N_2} + \cdots \frac{n_k}{N_k} = 1 \quad \text{or} \quad \sum_{j=1}^{j=k} \frac{n_j}{N_j} = 1$$

Tests cited by T. J. Dolan, et al., [Dolan, 1949] for progressively decreasing stress cycles show Miner's Rule to produce conservative results. On the other hand, tests reported in the same publication show the rule produces non-conservative results for progressively increasing stress cycles. Sines (1959) states, that for random stress amplitudes, Equation A1 gives "reasonably accurate" results. Stress fluctuations encountered in practice are usually random in nature, and under these conditions, the linear law appears to give predictions in the right range. Typical surge pressures in pipelines produce a dampened wave with progressively decreasing stress cycles. Therefore, Miner's Rule will produce reasonably accurate results - but on the conservative side.

Kirby (1980) reported that surge events in pressurized sewer systems typically have the appearance of the wave shown in Figure A1. This is essentially a dampened sine wave. For the case shown, the amplitude of the tenth cycle is between three and four percent of the initial cycle. The amplitudes of cycles after the tenth cycle are small and their influence on the cyclic life is negligible and can be disregarded.

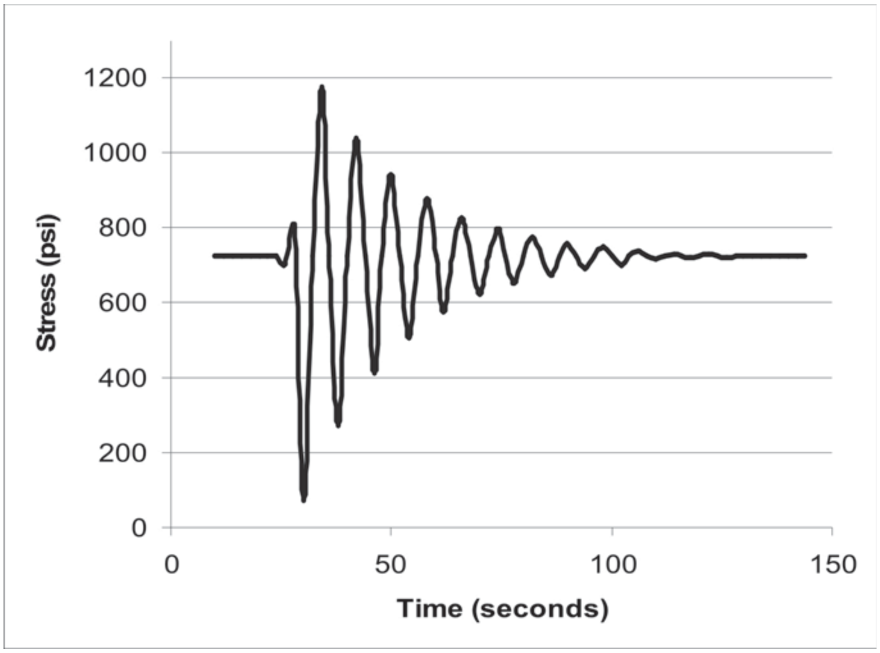


Figure A1: Pump Pressure Wave [Kirby, 1980]

Applying Miner's Rule

Table A1 gives the stress amplitudes for the first ten cycles in Figure A1. The average stress in Figure A1 is 725 psi. The cycles to failure for each average stress / stress amplitude combination are also shown and were derived from Figure 1. The method for using Figure 1 to determine $\mathbf{N_i}$ was presented in the section titled "Cyclic Check, Step-by-Step" as part of the PVC pressure pipe design example.

In the example in this Appendix, **k** is equal to ten. Moreover, every time there is a primary cycle (which has the subscript of one), the complex pressure wave has nine additional cycles (each having its own subscript ranging from two to ten). This known information is captured in Equation A2.

EQUATION A2

$$\mathbf{n_1 = n_2 = n_3 = n_4 = n_5 = n_6 = n_7 = n_8 = n_9 = n_{10} = n}$$

The design objective is to solve for **n** and determine the effect that the smaller cycles that accompany every primary cycle has on the pipe's cyclic life.

Table A1
Cycles to Failure for the Ten Cycles in Figure A1

Cycle Number (n_i)	Stress Amplitude (psi)	Average Stress (psi)	Cycles to Failure from Figure 1 (N_i)
1	652	725	1.4E+06
2	452	725	4.0E+06
3	313	725	1.5E+07
4	217	725	3.0E+07
5	151	725	6.0E+07
6	104	725	1.0E+08
7	72	725	1.4E+08
8	50	725	1.9E+08
9	35	725	2.3E+08
10	24	725	2.5E+08

The information known in Equation A2 allows Equation A1 to be further simplified as shown below in Equation A3.

EQUATION A3

$$\frac{n}{N_1}+\frac{n}{N_2}+\cdots\frac{n}{N_{10}}=1 \quad \text{or} \quad \sum_{j=1}^{j=10}\frac{n}{N_j}=1 \quad \text{or} \quad n\left(\sum_{j=1}^{j=10}\frac{1}{N_j}\right)=1$$

Equation A3 is solved for **n**. The result is shown below in Equation A4.

EQUATION A4

$$n=\frac{1}{\displaystyle\sum_{j=1}^{j=10}\frac{1}{N_j}}$$

The summation in the denominator of Equation A4 is calculated in Table A2.

Table A2
Application of Miner's Rule for Surge Event of Figure A1

Cycle Number (subscript i)	Cycles to Failure from Figure 1 (N_i)	$1 / N_i$
1	1.4E+06	7.14E-07
2	4.0E+06	2.50E-07
3	1.5E+07	6.67E-08
4	3.0E+07	3.33E-08
5	6.0E+07	1.67E-08
6	1.0E+08	1.00E-08
7	1.4E+08	7.14E-09
8	1.9E+08	5.26E-09
9	2.3E+08	4.35E-09
10	2.5E+08	4.00E-09
		$\sum_{j=1}^{j=10} \frac{1}{N_j} = 1.11E-06$

As shown in Equation A4, the value for **n** is the inverse of the summation calculated in the last row of Table A2. Thus, **n** is equal to 9.0 x 10^5 (1 ÷ 1.11E - 6).

For the primary wave, which has an average stress of 725 psi and a stress amplitude of 652 psi, the cycles to failure ($\mathbf{N_1}$) is 1.4 x 10^6. When each primary wave is immediately followed by nine additional - but smaller - waves as shown in Figure A1, the pipe's cyclic life is reduced to 9.0 x 10^5. Applying Miner's Rule shows that the secondary and tertiary waves that accompany the primary pressure wave may be represented as 0.55 primary cycles (1.4 x 10^6 ÷ 9.0 x 10^5 = 1.55). The use of this multiplier was demonstrated in the section titled "Reflected Pressure Waves" in the discussion on additional design considerations in the body of this report.

The fatigue of this system from the complex pressure wave shown in Figure A1 serves as an upper bound. The 1.55 multiplier may be used with confidence for analysis.

Please note that a properly designed system will not experience surge pressures with amplitudes of this magnitude. However, if such conditions are present in a system, the above procedure may be used to determine if corrective actions in the system are necessary to ensure a sufficient design life.

References Cited In Appendix A

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APPENDIX B

List of Equations

Equation 1
page 2

$$V = 1.318\,C\left(R_H\right)^{0.63}\left(S\right)^{0.54}$$

Equation 2
page 4

$$SDR = DR = D_o \div t_{min}$$

Equation 3
page 5

$$P_{peak} = P_{op} + V(P_s')$$

Equation 4
page 6

$$\sigma_{avg} = \frac{\left(P_{max} + P_{min}\right)\left(DR - 1\right)}{4}$$

Equation 5
page 6

$$\sigma_{amp} = \frac{\left(P_{max} - P_{min}\right)\left(DR - 1\right)}{4}$$

Equation 6
page 7

C′ = (N cycles/day) (365 days/year) (Design Life in Years)

Equation 7
page 8

$$D_i = D_o - 2t'$$

Equation 8
page 8

$$A_x = \frac{(\pi \div 4)(D_i)^2}{144}$$

Equation 9
page 8

$$V = Q \div A_x$$

Equation A1
page 21

$$\frac{n_1}{N_1} + \frac{n_2}{N_2} + \cdots \frac{n_k}{N_k} = 1 \quad \text{or} \quad \sum_{j=1}^{j=k} \frac{n_j}{N_j} = 1$$

Equation A2
page 22

$$n_1 = n_2 = n_3 = n_4 = n_5 = n_6 = n_7 = n_8 = n_9 = n_{10} = n$$

Equation A3
page 23

$$\frac{n}{N_1} + \frac{n}{N_2} + \cdots \frac{n}{N_{10}} = 1 \quad \text{or} \quad \sum_{j=1}^{j=10} \frac{n}{N_j} = 1 \quad \text{or} \quad n\left(\sum_{j=1}^{j=10} \frac{1}{N_j}\right) = 1$$

Equation A4
page 23

$$n = \frac{1}{\sum_{j=1}^{j=10} \frac{1}{N_j}}$$

APPENDIX C

Average Inside Diameters

The average inside diameters listed in this Appendix were calculated using Equation 7.

Table C1
ASTM D2241, IPS

Nominal Diameter (in)	Average OD (in)	SDR 17 Average ID (ft)	SDR 21 Average ID (ft)	SDR 26 Average ID (ft)	SDR 32.5 Average ID (ft)	SDR 41 Average ID (ft)
4	4.500	0.328	0.337	0.344	0.351	0.356
5	5.563	0.406	0.417	0.426	0.433	0.440
6	6.625	0.483	0.496	0.507	0.516	0.523
8	8.625	0.629	0.646	0.660	0.672	0.682
10	10.750	0.784	0.806	0.823	0.837	0.850
12	12.750	0.930	0.955	0.976	0.993	1.008
14	14.000	1.021	1.049	1.072	1.091	1.106
16	16.000	1.167	1.199	1.225	1.246	1.264
18	18.000	1.313	1.349	1.378	1.402	1.422
20	20.000	1.459	1.498	1.531	1.558	1.580
24	24.000	1.751	1.798	1.837	1.870	1.897
30	30.000	2.188	2.248	2.296	2.337	2.371
36	36.000	2.626	2.697	2.755	2.804	2.845

Table C2
AWWA C900, CIOD

Nominal Diameter (in)	Average OD (in)	DR 14 Average ID (ft)	DR 18 Average ID (ft)	DR 25 Average ID (ft)
4	4.800	0.339	0.353	0.366
6	6.900	0.488	0.507	0.526
8	9.050	0.640	0.665	0.690
10	11.100	0.785	0.816	0.847
12	13.200	0.933	0.971	1.007

Table C3
AWWA C905 CIOD

Nominal Diameter (in)	Average OD (in)	DR 18 Average ID (ft)	SDR 21 Average ID (ft)	DR 25 Average ID (ft)	SDR 32.5 Average ID (ft)	SDR 41 Average ID (ft)
14	15.300	1.125	xxxxx	1.167	1.192	1.209
16	17.400	1.279	xxxxx	1.327	1.355	1.375
18	19.500	1.434	1.461	1.487	1.519	1.541
20	21.600	1.588	1.618	1.647	1.683	1.707
24	25.800	1.897	1.933	1.968	2.010	2.039
30	32.000	xxxxx	2.397	2.441	2.493	2.529
36	38.300	xxxxx	2.869	2.921	2.984	3.027
42	44.500	xxxxx	xxxxx	xxxxx	3.475	3.522
48	50.800	xxxxx	xxxxx	xxxxx	3.967	4.021

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An American National Standard

Standard Practice for Underground Installation of Thermoplastic Pipe for Sewers and Other Gravity-Flow Applications[1]

This standard is issued under the fixed designation D2321; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This standard has been approved for use by agencies of the Department of Defense.

1. Scope*

1.1 This practice provides recommendations for the installation of buried thermoplastic pipe used in sewers and other gravity-flow applications. These recommendations are intended to ensure a stable underground environment for thermoplastic pipe under a wide range of service conditions. However, because of the numerous flexible plastic pipe products available and the inherent variability of natural ground conditions, achieving satisfactory performance of any one product may require modification to provisions contained herein to meet specific project requirements.

1.2 The scope of this practice necessarily excludes product performance criteria such as minimum pipe stiffness, maximum service deflection, or long term strength. Thus, it is incumbent upon the product manufacturer, specifier, or project engineer to verify and assure that the pipe specified for an intended application, when installed according to procedures outlined in this practice, will provide a long term, satisfactory performance according to criteria established for that application. A commentary on factors important in achieving a satisfactory installation is included in Appendix X1.

NOTE 1—Specific paragraphs in the appendix are referenced in the body of this practice for informational purposes.

NOTE 2—The following ASTM standards may be found useful in connection with this practice: Practice D420, Test Method D1556, Method D2216, Specification D2235, Test Method D2412, Specification D2564, Practice D2657, Practice D2855, Test Methods D2922, Test Method D3017, Practice F402, Specification F477, Specification F545, and Specification F913.

NOTE 3—Most Plumbing Codes and some Building Codes have provisions for the installation of underground "building drains and building sewers." See them for plumbing piping applications.

1.3 *Units*—The values stated in inch-pound units are to be regarded as standard. The values given in parentheses are mathematical conversions to SI units that are provided for information only and are not considered standard.

1.4 *This standard does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*[2]

D8 Terminology Relating to Materials for Roads and Pavements[3]

D420 Guide to Site Characterization for Engineering Design and Construction Purposes

D653 Terminology Relating to Soil, Rock, and Contained Fluids

D698 Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12 400 ft-lbf/ft^3(600 kN-m/m^3))

D1556 Test Method for Density and Unit Weight of Soil in Place by Sand-Cone Method

D2216 Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass

D2235 Specification for Solvent Cement for Acrylonitrile-Butadiene-Styrene (ABS) Plastic Pipe and Fittings

D2412 Test Method for Determination of External Loading Characteristics of Plastic Pipe by Parallel-Plate Loading

D2487 Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)

D2488 Practice for Description and Identification of Soils (Visual-Manual Procedure)

D2564 Specification for Solvent Cements for Poly(Vinyl Chloride) (PVC) Plastic Piping Systems

D2657 Practice for Heat Fusion Joining of Polyolefin Pipe and Fittings

D2855 Practice for Making Solvent-Cemented Joints with Poly(Vinyl Chloride) (PVC) Pipe and Fittings

[1] This practice is under the jurisdiction of ASTM Committee F17 on Plastic Piping Systems and is the direct responsibility of Subcommittee F17.62 on Sewer.

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[2] For referenced ASTM standards, visit the ASTM website, www.astm.org, or contact ASTM Customer Service at service@astm.org. For *Annual Book of ASTM Standards* volume information, refer to the standard's Document Summary page on the ASTM website.

[3] Withdrawn. The last approved version of this historical standard is referenced on www.astm.org.

***A Summary of Changes section appears at the end of this standard.**

D2922 Test Methods for Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth)[3]
D3017 Test Method for Water Content of Soil and Rock in Place by Nuclear Methods (Shallow Depth)
D3839 Guide for Underground Installation of "Fiberglass" (Glass-FiberReinforced Thermosetting-Resin) Pipe
D4318 Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils
F402 Practice for Safe Handling of Solvent Cements, Primers, and Cleaners Used for Joining Thermoplastic Pipe and Fittings
F412 Terminology Relating to Plastic Piping Systems
F477 Specification for Elastomeric Seals (Gaskets) for Joining Plastic Pipe
F545 Specification for PVC and ABS Injected Solvent Cemented Plastic Pipe Joints[3]
F913 Specification for Thermoplastic Elastomeric Seals (Gaskets) for Joining Plastic Pipe
F1668 Guide for Construction Procedures for Buried Plastic Pipe

2.2 *AASHTO Standard:*[4]
AASHTO M145 Classification of Soils and Soil Aggregate Mixtures

3. Terminology

3.1 *General—Definitions used in this practice are in accordance with Terminologies F412 and D8 and Terminology D653 unless otherwise indicated.*

3.1.1 Terminology D653 definitions used in this standard:

3.1.2 *compaction curve (Proctor curve) (moisture-density curve)*—the curve showing the relationship between the dry unit weight (density) and the water content of a soil for a given compactive effort.

3.1.3 *optimum water content* —the water content at which a soil can be compacted to a maximum dry unit weight by a given compactive effort.

3.1.4 *percent compaction*—the ratio, expressed as a percentage, of: *(1)* dry unit weight of a soil, to *(2)* maximum unit weight obtained in a laboratory compaction test.

3.1.5 *maximum unit weight*—the dry unit weight defined by the peak of a compaction curve.

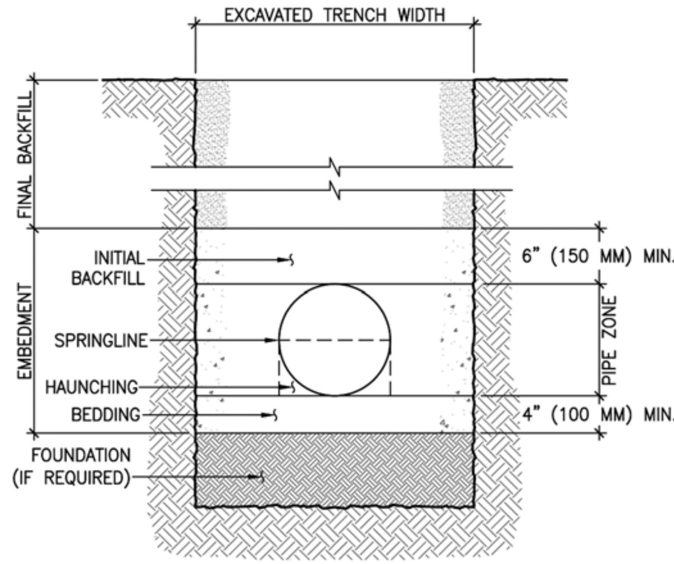
3.2 *Definitions of Terms Specific to This Standard:*

3.2.1 *foundation, bedding, haunching, initial backfill, final backfill, pipe zone, excavated trench width*—See Fig. 1 for meaning and limits, and trench terminology.

3.2.2 *aggregate*—a granular material of mineral composition such as sand, gravel, shell, slag or crushed stone (see Terminology D8).

3.2.3 *deflection*—any change in the inside diameter of the pipe resulting from installation and imposed loads. Deflection may be either vertical or horizontal and is usually reported as a percentage of the base (undeflected) inside pipe diameter.

3.2.4 *engineer*—the engineer in responsible charge of the work or his duly recognized or authorized representative.



* See 7.6 Minimum Cover

FIG. 1 Trench Cross Section

3.2.5 *manufactured aggregates*—aggregates such as slag that are products or byproducts of a manufacturing process, or natural aggregates that are reduced to their final form by a manufacturing process such as crushing.

3.2.6 *modulus of soil reaction (E')*—an empirical value used in the Iowa deflection formula that defines the stiffness of the soil embedment around a buried pipe

3.2.7 *open-graded aggregate*—an aggregate that has a particle size distribution such that, when it is compacted, the voids between the aggregate particles, expressed as a percentage of the total space occupied by the material, are relatively large.

3.2.8 *processed aggregates*—aggregates that are screened, washed, mixed, or blended to produce a specific particle size distribution.

3.2.9 *secant constrained soil modulus (M_s)*—- a value for soil stiffness determined as the secant slope of the stress-strain curve of a one-dimensional compression test; M_s can be used in place of *E'* in the Iowa deflection formula.

3.2.10 *standard proctor density*—the maximum dry unit weight of soil compacted at optimum moisture content, as obtained by laboratory test in accordance with Test Methods D698.

4. Significance and Use

4.1 This practice is for use by designers and specifiers, installation contractors, regulatory agencies, owners, and inspection organizations who are involved in the construction of sewers and other gravity-flow applications that utilize flexible thermoplastic pipe. As with any standard practice, modifications may be required for specific job conditions or for special local or regional conditions. Recommendations for inclusion of this practice in contract documents for a specific project are given in Appendix X2.

5. Materials

5.1 *Classification*—Soil types used or encountered in burying pipes include those classified in Table 1 and natural,

[4] Available from American Association of State Highway and Transportation Officials (AASHTO), 444 N. Capitol St., NW, Suite 249, Washington, DC 20001, http://www.transportation.org.

TABLE 1 Soil Classification Chart (see Classification D2487)

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests[A]					Soil Classification	
					Group Symbol	Group Name[B]
Coarse-Grained Soils	gravels	clean gravels	$C \geq 4$ and $1 \leq Cc \leq 3$[C]		GW	well-graded gravel[D]
More than 50% retained on No. 200 sieve	more than 50% of coarse fraction retained on No. 4 sieve	less than 5% of fines[E]	$Cu < 4$ and/or $1 > Cc > 3$[C]		GP	poorly graded gravel[D]
		gravels with more than 12 % fines[E]	Fines classify as ML or MH		GM	silty gravel[DFG]
			Fines classify as CL or CH		GC	clayey gravel[DFG]
	sands	clean sands	$Cu \geq 6$ and $1 \leq Cc \leq 3$[C]		SW	well-graded sand[H]
	50% or more of coarse fraction passes on No. 4 sieve	less than 5% fines[I]	$Cu < 6$ and/or $1 > Cc > 3$[C]		SP	poorly graded sand[H]
		sand with fines	Fines cLassify as ML or MH		SM	silty sand[FGH]
		more than 12 % fines[I]	Fines classify as CL or CH		SC	clayey sand[FGH]
Fine-Grained Soils	silts and clays	inorganic	PI > 7 and plots on or above “A” line[J]		CL	lean clay[KLM]
50% or more passes the No. 200 sieve	liquid limit less than 50		PI < 4 and plots below “A” line[J]		ML	silt[KLM]
		organic	Liquid Limit-Oven dried	<0.75	OL	organic clay[KLMN]
			Liquid Limit-Not dried			organic silt[KLMO]
	silts and clays	inorganic	PI plots on or above “A” line		CH	fat clay[KLM]
	liquid limit 50 or more		Plots below “A” line		MH	elastic silt[KLM]
		organic	Liquid Limit-Oven Dried	<0.75	OH	organic clay[KLMP]
			Liquid Limit-Not Dried			organic silt[KLMQ]
Highly organic soils	primarily organic matter, dark in color, and organic odor				PT	peat

[A] Based on the material passing the 3-in. (75-mm) sieve.
[B] If field sample contained cobbles or boulders, or both, add “with cobbles or boulders, or both” to group name.
[C] $Cu = D_{60}/D_{10}$

$$Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$$

[D] If soil contains ≥15 % sand, add “with sand” to group name.
[E] Gravels with 5 to 12 % fines require dual symbols:
GW-GM well-graded gravel with silt:
GW-GC well-graded gravel with clay
GP-GM poorly graded gravel with silt
GP-GC poorly graded gravel with clay
[F] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.
[G] If fines are organic, add “with organic fines” to group name.
[H] If soil contains ≥15 % gravel, add “with gravel” to group name.
[I] Sands with 5 to 12 % fines require dual symbols:
SW-SM well-graded sand with silt
SW-SC well-graded sand with clay
SP-SM poorly graded sand with silt
SP-SC poorly graded sand with clay
[J] If Atterberg limits plot in hatched area, soil is a CL-ML, silty clay (see Test Method D4318).
[K] If soil contains 15 to 29 % plus No. 200, add “with sand” or “with gravel,” whichever is predominant.
[L] If soil contains ≥ 30 % plus No. 200, predominantly sand, add “sandy” to group name.
[M] If soil contains ≥ 30 % plus No. 200, predominantly gravel, add “gravelly” to group name.
[N] PI ≥ 4 and plots on or above “A” line.
[O] PI < 4 or plots below “A” line.
[P] PI plots on or above “A” line.
[Q] PI plots below “A” line.

manufactured, and processed aggregates. The soil classifications are grouped into soil classifications in Table 2 based on the typical soil stiffness when compacted. Class I indicates a soil that generally provides the highest soil stiffness at any given percent compaction, and provides a given soil stiffness with the least compactive effort. Each higher-number soil class provides successively less soil stiffness at a given percent compaction and requires greater compactive effort to provide a given level of soil stiffness

NOTE 4—See Practices D2487 and D2488 for laboratory and field visual-manual procedures for identification of soils.

NOTE 5—Processed materials produced for highway construction, including coarse aggregate, base, subbase, and surface coarse materials, when used for foundation, embedment, and backfill, should be categorized in accordance with this section and Table 1 in accordance with particle size and gradation.

5.2 *Installation and Use*—Table 3 provides recommendations on installation and use based on soil classification and location in the trench. Soil Classes I to IV should be used as recommended in Table 3. Soil Class V, including clays and silts with liquid limits greater than 50, organic soils, and frozen soils, shall be excluded from the pipe-zone embedment.

5.2.1 *Class I*— Class I materials provide maximum stability and pipe support for a given percent compaction due to the low content of sand and fines. With minimum effort these materials can be installed at relatively high-soil stiffnesses over a wide range of moisture contents. In addition, the high permeability of Class I materials may aid in the control of water, and these materials are often desirable for embedment in rock cuts where water is frequently encountered. However, when ground-water flow is anticipated, consideration should be given to the potential for migration of fines from adjacent materials into the open-graded Class I materials. (See X1.8.)

5.2.2 *Class II*—Class II materials, when compacted, provide a relatively high level of pipe support; however, open-graded groups may allow migration and the sizes should be checked for compatibility with adjacent material. (See X1.8.)

5.2.3 *Class III*— Class III materials provide less support for a given percent compaction than Class I or Class II materials. Higher levels of compactive effort are required and moisture content must be near optimum to minimize compactive effort and achieve the required percent compaction. These materials provide reasonable levels of pipe support once proper percent compaction is achieved.

5.2.4 *Class IV*—Class IV materials require a geotechnical evaluation prior to use. Moisture content must be near optimum to minimize compactive effort and achieve the required percent compaction. Properly placed and compacted, Class IV materials can provide reasonable levels of pipe support;

TABLE 2 Soil Classes

Soil Group[A,B]	Soil Class	American Association of State Highway and Transportation Officials (AASHTO) Soil Groups[C]
Crushed rock, angular[D]: 100% passing 1-1/2in. sieve, </=15 % passing #4 sieve, </= 25 % passing 3/8in. sieve and </= 12 % passing #200 sieve	Class I	...
Clean, coarse grained soils: SW, SP, GW, GP or any soil beginning with one of these symbols with </=12 % passing #200 sieve[E,F]	Class II	A1,A3
Coarse grained soils with fines: GM, GC, SM, SC, or any soil beginning with one of these symbols, containing > 12 % passing #200 sieve; Sandy or gravelly fine-grained soils: CL, ML, or any soil beginning with one of these symbols, with >/= 30 % retained on #200 sieve	Class III	A-2-4, A-2-5, A-2-6, or A-4 or A-6 soils with more than 30% retained on #200 sieve
Fine-grained soils: CL, ML, or any soil beginning with one of these symbols, with <30 % retained on #200 sieve	Class IV	A-2-7, or A-4, or A-6 soils with 30% or less retained on #200 sieve
MH, CH, OL, OH, PT	Class V Not for use as embedment	A5, A7

[A] See Classification D2487, Standard Classification of Soils for Engineering Purposes (Unified Soil Classification System).
[B] Limits may be imposed on the soil group to meet project or local requirements if the specified soil remains within the group. For example, some project applications require a Class I material with minimal fines to address specific structural or hydraulic conditions and the specification may read "Use Class I soil with a maximum of 5% passing the #200 sieve."
[C] AASHTO M145, Classification of Soils and Soil Aggregate Mixtures.
[D] All particle faces shall be fractured.
[E] Materials such as broken coral, shells, and recycled concrete, with ≤ =12% passing a No. 200 sieve, are considered to be Class II materials. These materials should only be used when evaluated and approved by the Engineer
[F] Uniform fine sands (SP) with more than 50% passing a No. 100 sieve (0.006 in., 0.15 mm) are very sensitive to moisture and should not be used as backfill unless specifically allowed in the contract documents. If use of these materials is allowed, compaction and handling procedures should follow the guidelines for Class III materials.

TABLE 3 Recommendations for Installation and Use of Soils and Aggregates for Foundation and Pipe-Zone Embedment

Soil Class[A]	Class I[B]	Class II	Class III	Class IV
General Recommendations and Restrictions	Acceptable and common where no migration is probable or when combined with a geotextile filter media. Suitable for use as a drainage blanket and under drain where adjacent material is suitably graded or when used with a geotextile filter fabric (see X1.8).	Where hydraulic gradient exists check gradation to minimize migration. Clean groups are suitable for use as a drainage blanket and underdrain (see Table 2). Uniform fine sands (SP) with more than 50 % passing a #100 sieve (0.006 in., 0.15 mm) behave like silts and should be treated as Class IV soils.	Do not use where water conditions in trench prevent proper placement and compaction. Not recommended for use with pipes with stiffness of 9 psi or less	Difficult to achieve high-soil stiffness. Do not use where water conditions in trench prevent proper placement and compaction. Not recommended for use with pipes with stiffness of 9 psi or less
Foundation	Suitable as foundation and for replacing over-excavated and unstable trench bottom as restricted above.	Suitable as foundation and for replacing over-excavated and unstable trench bottom as restricted above. Install and compact in 12 in. (300 mm) maximum layers	Suitable for replacing over-excavated trench bottom as restricted above. Install and compact in 6 in. (150 mm) maximum layers	Suitable for replacing over-excavated trench bottom as restricted above. Install and compact in 6-in (150 mm) maximum layers
Pipe Embedment	Suitable as restricted above. Work material under pipe to provide uniform haunch support.	Suitable as restricted above. Work material under pipe to provide uniform haunch support.	Suitable as restricted above. Difficult to place and compact in the haunch zone.	Suitable as restricted above. Difficult to place and compact in the haunch zone.
Embedment Compaction: Min Recommended Percent Compaction, SPD[D]	See Note[C]	85 % (SW and SP soils) For GW and GP soils see Note[E]	90 %	95 %
Relative Compactive Effort Required to Achieve Minimum Percent Compaction	low	moderate	high	very high
Compaction Methods	vibration or impact	vibration or impact	impact	impact
Required Moisture Control	none	none	Maintain near optimum to minimize compactive effort	Maintain near optimum to minimize compactive effort

[A] Class V materials are unsuitable as embedment. They may be used as final backfill as permitted by the engineer.
[B] Class I materials have higher stiffness than Class II materials, but data on specific soil stiffness of placed, uncompacted Class I materials can be taken equivalent to Class II materials compacted to 95% of maximum standard Proctor density (SPD95), and the soil stiffness of compacted Class I materials can be taken equivalent to Class II materials compacted to 100% of maximum standard Proctor density (SPD100). Even if placed uncompacted (that is, dumped), Class I materials should always be worked into the haunch zone to assure complete placement.
[C] Suitable compaction typically achieved by dumped placement (that is, uncompacted but worked into haunch zone to ensure complete placement).
[D] SPD is standard Proctor density as determined by Test Method D698.
[E] Place and compact GW and GP soils with at least two passes of compaction equipment.

however, these materials may not be suitable under high fills, surface-applied wheel loads, or under high-energy-level vibratory compactors and tampers. Do not use where water conditions in the trench may prevent proper placement and compaction.

NOTE 6—The term "high energy level vibratory compactors and tampers" refers to compaction equipment that might deflect or distort the pipe more than permitted by the specifications or the manufacturer.

5.2.5 *Class V*—Class V materials should be excluded from pipe-zone embedment.

5.3 *Moisture Content of Embedment Materials*—The moisture content of embedment materials must be controlled to permit placement and compaction to required levels. For soils with low permeability (that is, Class III and Class IV and some borderline Class II soils), moisture content is normally controlled to ± 3 % of optimum (see Test Method D698). The practicality of obtaining and maintaining the required limits on moisture content is an important criterion for selecting materials, since failure to achieve required percent compaction, especially in the pipe zone embedment, may result in excessive deflection.

5.4 *Maximum Particle Size*—Maximum particle size for embedment is limited to material passing a 1½-in. (37.5-mm) sieve (see Table 2). To enhance placement around small diameter pipe and to prevent damage to the pipe wall, a smaller maximum size may be required (see X1.9). When final backfill contains rocks, cobbles, etc., the engineer may require greater initial backfill cover levels (see Fig. 1).

6. Trench Excavation

6.1 *General*—Procedures for trench excavation that are especially important in flexible thermoplastic pipe installations are given herein.

6.1.1 *Excavation*—Excavate trenches to ensure that sides will be stable under all working conditions. Slope trench walls or provide supports in conformance with all local and national standards for safety. Open only as much trench as can be safely maintained by available equipment. Backfill all trenches as soon as practicable, but not later than the end of each working day.

6.2 *Water Control*—Do not lay or embed pipe in standing or running water. At all times prevent runoff and surface water from entering the trench.

6.2.1 *Ground Water*—When groundwater is present in the work area, dewater to maintain stability of in-situ and imported materials. Maintain water level below pipe bedding and foundation to provide a stable trench bottom. Use, as appropriate, sump pumps, well points, deep wells, geofabrics, perforated underdrains, or stone blankets of sufficient thickness to remove and control water in the trench. When excavating while depressing ground water, ensure the ground water is below the bottom of cut at all times to prevent washout from behind sheeting or sloughing of exposed trench walls. Maintain control of water in the trench before, during, and after pipe installation, and until embedment is installed and sufficient backfill has been placed to prevent flotation of the pipe. To preclude loss of soil support, employ dewatering methods that minimize removal of fines and the creation of voids in in-situ materials.

6.2.2 *Running Water*—Control running water emanating from drainage of surface or ground water to preclude undermining of the trench bottom or walls, the foundation, or other zones of embedment. Provide dams, cutoffs or other barriers periodically along the installation to preclude transport of water along the trench bottom. Backfill all trenches after the pipe is installed to prevent disturbance of pipe and embedment.

6.2.3 *Materials for Water Control*—Use suitably graded materials in foundation or bedding layers or as drainage blankets for transport of running water to sump pits or other drains. Use well graded materials, along with perforated underdrains, to enhance transport of running water, as required. Select the gradation of the drainage materials to minimize migration of fines from surrounding materials (see X1.8).

6.3 *Minimum Trench Width*—Where trench walls are stable or supported, provide a width sufficient, but no greater than necessary, to ensure working room to properly and safely place and compact haunching and other embedment materials. The space between the pipe and trench wall must be wider than the compaction equipment used in the pipe zone. Minimum width shall be not less than the greater of either the pipe outside diameter plus 16 in. (400 mm) or the pipe outside diameter times 1.25, plus 12 in. (300 mm). In addition to safety considerations, trench width in unsupported, unstable soils will depend on the size and stiffness of the pipe, stiffness of the embedment and in-situ soil, and depth of cover (see X1.10). Specially designed equipment may enable the satisfactory installation and embedment of pipe in trenches narrower than specified above. If it is determined that the use of such equipment provides an installation consistent with the requirements of this standard, minimum trench widths may be reduced, as approved by the engineer.

6.4 *Support of Trench Walls*—When supports such as trench sheeting, trench jacks, trench shields or boxes are used, ensure that support of the pipe and its embedment is maintained throughout installation. Ensure that sheeting is sufficiently tight to prevent washing out of the trench wall from behind the sheeting. Provide tight support of trench walls below viaducts, existing utilities, or other obstructions that restrict driving of sheeting.

6.4.1 *Supports Left in Place*—Unless otherwise directed by the engineer, sheeting driven into or below the pipe zone should be left in place to preclude loss of support of foundation and embedment materials. When top of sheeting is to be cut off, make cut 1.5 ft (0.5 m) or more above the crown of the pipe. Leave rangers, whalers, and braces in place as required to support cutoff sheeting and the trench wall in the vicinity of the pipe zone. Timber sheeting to be left in place is considered a permanent structural member and should be treated against biological degradation (for example, attack by insects or other biological forms) as necessary, and against decay if above ground water.

NOTE 7—Certain preservative and protective compounds may react adversely with some types of thermoplastics, and their use should be avoided in proximity of the pipe material.

6.4.2 *Movable Trench Wall Supports*—Do not disturb the installed pipe and its embedment when using movable trench boxes and shields. Movable supports should not be used below the top of the pipe zone unless approved methods are used for maintaining the integrity of embedment material. Before moving supports, place and compact embedment to sufficient depths to ensure protection of the pipe. As supports are moved, finish placing and compacting embedment.

6.4.3 *Removal of Trench Wall Support*—If the engineer permits the use of sheeting or other trench wall supports below the pipe zone, ensure that pipe and foundation and embedment materials are not disturbed by support removal. Fill voids left on removal of supports and compact all material as required.

6.5 *Rock or Unyielding Materials in Trench Bottom*—If ledge rock, hard pan, shale, or other unyielding material, cobbles, rubble or debris, boulders, or stones larger than 1.5 in. (40 mm) are encountered in the trench bottom, excavate a minimum depth of 6 in. (150 mm) below the pipe bottom and replace with proper embedment material (see 7.2.1).

7. Installation

7.1 *General*—Recommendations for use of the various types of materials classified in Section 5 and Table 2 for foundation, bedding, haunching and backfills, are given in Table 3.

NOTE 8—Installation of pipe in areas where significant settlement may be anticipated, such as in backfill adjacent to building foundations, and in sanitary landfills, or in other highly unstable soils, require special engineering and are outside the scope of this practice.

7.2 *Trench Bottom*—Install foundation and bedding as required by the engineer according to conditions in the trench bottom. Provide a firm, stable, and uniform bedding for the pipe barrel and any protruding features of its joint. Provide a minimum of 4 in. (100 mm) of bedding unless otherwise specified.

7.2.1 *Rock and Unyielding Materials*—When rock or unyielding material is present in the trench bottom, install a cushion of bedding, of 6 in. (150 mm) minimum thickness, below the bottom of the pipe.

7.2.2 *Unstable Trench Bottom*—Where the trench bottom is unstable or shows a "quick'' tendency, excavate to a depth as required by the engineer and replace with a foundation of Class I or Class II material. Use a suitably graded material where conditions may cause migration of fines and loss of pipe support (see X1.8). Place and compact foundation material in accordance with Table 3. For severe conditions, the engineer may require a special foundation such as piles or sheeting capped with a concrete mat. Control of quick and unstable trench bottom conditions may be accomplished with the use of appropriate geofabrics.

7.2.3 *Localized Loadings*—Minimize localized loadings and differential settlement wherever the pipe crosses other utilities or subsurface structures, or whenever there are special foundations such as concrete capped piles or sheeting. Provide a cushion of bedding between the pipe and any such point of localized loading.

7.2.4 *Over-Excavation*—If the trench bottom is over-excavated below intended grade, fill the over-excavation with compatible foundation or bedding material and compact as recommended in Table 3.

7.2.5 *Sloughing*—If trench sidewalls slough off during any part of excavating or installing the pipe, remove all sloughed and loose material from the trench.

7.3 *Location and Alignment*—Place pipe and fittings in the trench with the invert conforming to the required elevations, slopes, and alignment. Provide bell holes in pipe bedding, no larger than necessary, in order to ensure uniform pipe support. Fill all voids under the bell by working in bedding material. In special cases where the pipe is to be installed to a curved alignment, maintain angular "joint deflection'' (axial alignment) or pipe bending radius, or both, within acceptable design limits.

7.4 *Jointing*—Comply with manufacturer's recommendations for assembly of joint components, lubrication, and making of joints. When pipe laying is interrupted, secure piping against movement and seal open ends to prevent the entrance of water, mud, or foreign material.

7.4.1 *Elastomeric Seal Joints*—Protect gaskets from harmful substances such as dust and grit, solvents, and petroleum-based greases and oils. Do not store gaskets close to electrical equipment that produces ozone. Some gaskets may need to be protected from sunlight (consult the manufacturer). Mark, or verify that pipe ends are marked, to indicate insertion stop position, and ensure that pipe is inserted into pipe or fitting bells to this mark. Push spigot into bell using methods recommended by the manufacturer, keeping pipe true to line and grade. Protect the end of the pipe while inserting the spigot into the bell and do not use excessive force that may result in over-assembled joints or dislodged gaskets. If full entry to the specified insertion depth is not achieved, disassemble and clean the joint and reassemble. Use only lubricant supplied or recommended for use by the pipe manufacturer. Do not exceed manufacturer's recommendations for angular "joint deflection'' (axial alignment).

7.4.2 *Solvent Cement Joints*—When making solvent cement joints, follow recommendations of both the pipe and solvent cement manufacturer. If full entry is not achieved, disassemble or remove and replace the joint. Allow freshly made joints to set for the recommended time before moving, burying, or otherwise disturbing the pipe.

7.4.3 *Heat Fusion Joints*—Make heat fusion joints in conformance with the recommendations of the pipe manufacturer. Pipe may be joined at ground surface and then lowered into position, provided it is supported and handled in a manner that precludes damage.

7.5 *Placing and Compacting Pipe Embedment*—Place embedment materials by methods that will not disturb or damage the pipe. Work in and tamp the haunching material in the area between the bedding and the underside of the pipe before placing and compacting the remainder of the embedment in the pipe zone. Follow recommendations for compaction given in Table 2. Do not permit compaction equipment to contact and damage the pipe. Use compaction equipment and techniques that are compatible with materials used and location in the trench (see X1.7). Before using heavy compaction or construction equipment directly over the pipe, place sufficient backfill to prevent damage, excessive deflections, or other disturbance of the pipe. See 7.6 for minimum cover.

7.5.1 *Percent Compaction of Embedment* —The Soil Class (from Table 2) and the required percent compaction of the embedment should be established by the engineer based on an evaluation of specific project conditions (see X1.6.2). The information in Table 3 will provide satisfactory embedment stiffness and is based on achieving an average modulus of soil reaction, E', of 1000 psi (or an appropriate equivalent constrained modulus, M_s).

7.5.2 *Consolidation by Watering*—Consolidation of cohesionless material by using water (jetting or puddling) should only be used under controlled conditions when approved by the engineer. At all times conform to the lift thicknesses and the compaction requirements given in Table 3.

7.6 *Minimum Cover*—To preclude damage to the pipe and disturbance to pipe embedment, a minimum depth of backfill above the pipe should be maintained before allowing vehicles or heavy construction equipment to traverse the pipe trench. The minimum depth of cover should be established by the engineer based on an evaluation of specific project conditions. In the absence of an engineering evaluation, the following minimum cover requirements should be used. For embedment materials installed in accordance with Table 3, provide cover (that is, depth of backfill above top of pipe) of at least 24 in. (0.6 m) or one pipe diameter (whichever is larger) for Class I embedment, and a cover of at least 36 in. (0.9 m) or one pipe diameter (whichever is larger) for Class II, III, and IV embedment, before allowing vehicles or construction equipment to traffic the trench surface, and at least 48 in. (1.2 m) of cover before using a hydrohammer for compaction. Do not use hydrohammer-type compactors unless approved by the engineer. Where construction loads may be excessive (for example, cranes, earth moving equipment, etc.), minimum cover shall be increased as determined by the engineer.

7.7 *Vertical Risers*—Provide support for vertical risers as commonly found at service connections, cleanouts, and drop manholes to preclude vertical or lateral movement. Prevent the direct transfer of thrust due to surface loads and settlement, and ensure adequate support at points of connection to main lines.

7.8 *Exposing Pipe for Making Service Line Connections*—When excavating for a service line connection, excavate material from above the top of the existing pipe before removing material from the sides of the pipe. Materials and percent compaction of service line embedment should conform to the specifications for the existing line, or with this practice, whichever is more stringent.

NOTE 9—Special construction techniques and considerations are required when more than one pipe is installed in the same or adjacent trenches, to ensure that the integrity of the embedment is maintained.

7.9 *Pipe Caps and Plugs*—Secure caps and plugs to the pipe to prevent movement and resulting leakage under test and service pressures.

7.10 *Manhole Connections*—Use flexible water stops, resilient connectors, or other flexible systems approved by the engineer to make watertight connections to manholes and other structures.

7.11 *Field Monitoring*—Compliance with contract documents with respect to pipe installation, including trench depth, grade, water conditions, foundation, embedment and backfill materials, joints, density of materials in place, and safety, should be monitored by the engineer at a frequency appropriate to project requirements. Leakage testing specifications, while not within the scope of this practice, should be made part of the specifications for plastic pipe installations, when applicable.

8. Inspection, Handling, and Storage

8.1 *Inspection*—Upon receipt, inspect each shipment of pipe and fittings for conformance to product specifications and contract documents, and check for damage. Reject nonconforming or damaged pipe, and remove from the job. If not returned to supplier, dispose of legally.

8.2 *Handling and Storage*—Handle and store pipe and fittings in accordance with recommendations of the manufacturer.

9. Keywords

9.1 backfill; bedding; compaction; embedment; haunching; migration; sewer pipe; soil stiffness; thermoplastic; underground installation

APPENDIXES

(Nonmandatory Information)

X1. COMMENTARY

X1.1 Those concerned with the service performance of a buried flexible pipe should understand factors that can affect this performance. Accordingly, key considerations in the design and execution of a satisfactory installation of buried flexible thermoplastic pipe that provided a basis for the development of this practice are given in this Appendix.

X1.2 *General*—Sub-surface conditions should be adequately investigated prior to construction, in accordance with Practice D420, as a basis for establishing requirements for foundation, embedment and backfill materials and construction methods. The type of pipe selected should be suited for the job conditions.

X1.3 *Load/Deflection Performance*—The thermoplastic pipes considered in this practice are classified as flexible conduits since in carrying load they deform (deflect) to develop support from the surrounding embedment. This interaction of pipe and soil provides a pipe-soil structure capable of supporting earth fills and surface live loads of considerable magnitude. The design, specification and construction of the buried flexible pipe system should recognize that embedment materials must be selected, placed and compacted so that pipe and soil act in concert to carry the applied loads without excessive strains from deflections or localized pipe wall distortions.

X1.4 *Pipe Deflection*—Pipe deflection is the diametral change in the pipe-soil system resulting from the process of installing the pipe (construction deflection), static and live loads applied to the pipe (load-induced deflection), and time dependent soil response (deflection lag). Construction and load induced deflections together constitute initial pipe deflection. Additional time dependent deflections are attributed primarily to changes in embedment and in-situ soils, and trench settlement. The sum of initial and time dependent deflections constitutes total deflection.

X1.4.1 *Construction Deflection*

Construction deflections are induced during the process of installing and embedding flexible pipe, even before significant earth and surface loads are applied. The magnitude of construction deflections depends on such factors as the method and extent of compaction of the embedment materials, type of embedment, water conditions in the trench, pipe stiffness, uniformity of embedment support, pipe out-of-roundness, and installation workmanship in general. These deflections may exceed the subsequent load-induced deflections. Compaction of the side fill may result in negative vertical deflections (that is, increases in pipe vertical diameter and decreases in horizontal diameter).

X1.4.2 *Load-Induced Deflection*

Load-induced deflections result from backfill loads and other superimposed loads that are applied after the pipe is embedded. Traditionally, typical soil-structure interaction equations such as the "Iowa Formula'', attributed to Spangler, or other methods have been used to calculate deflections resulting from these loads.

X1.4.3 *Initial Deflection*

Initial deflection is the deflection in the installed and backfilled pipe. It is the total of construction deflections and load-induced deflections.

X1.4.4 *Time Dependent Factors*

Time dependent factors include changes in soil stiffness in the pipe embedment zone and native trench soils, as well as loading changes due to trench settlement over time. These changes typically add to initial deflections; the time involved varies from a few days to several years depending on soil types, their placement, and initial compaction. Time dependent factors are traditionally accounted for by adjusting load-induced deflections by a deflection lag factor. Selection of a deflection lag factor is considered in design guides for buried flexible pipe.

X1.4.5 *Final Deflection*

Final deflection is the total long term deflection of the pipe. It consists of initial deflection adjusted for time dependent factors.

X1.5 *Deflection Criteria*—Deflection criteria are often set as limits for the design and acceptance of buried flexible pipe installation. Deflection limits for specific pipe systems may be derived from both structural and practical considerations. Structural considerations include pipe cracking, yielding, strength, strain, and local distortion. Practical considerations include such factors as flow requirements, clearance for inspection and cleaning, and maintenance of joint seals. Initial and final deflection limits should be based on available structural properties with suitable factors of safety applied.

NOTE X1.1—Some ASTM standard specifications for thermoplastic pipe, such as Specifications D3034, F679, F714, and F949, provide recommended limits for installed deflections.

NOTE X1.2—Deflections may not be indicative of strain levels arising from local distortions caused by non-uniform embedment stiffness or localized loadings. When local distortions may be significant, the engineer needs to establish methods for controlling and monitoring distortion levels.

X1.6 *Deflection Control*—Embedment materials should be selected, placed, and compacted so as to minimize total deflections and, in any event, to maintain installed deflections within specific limits. Methods of placement, compaction, and moisture control should be selected based on soil types given in Table 1 and Table 2 and on recommendations given in Table 3. The amount of load-induced deflection is primarily a function of the stiffness of the pipe and soil embedment system. Other factors that are important in obtaining deflection control are outlined below.

X1.6.1 *Embedment at Pipe Haunches*

Lack of adequate compaction of embedment material in the haunch zone can result in excessive deflection, since it is this material that supports the vertical loads applied to the pipe. A key objective during installation of flexible thermoplastic pipe (or any pipe) is to work in and compact embedment material under pipe haunches, to ensure complete contact with the pipe bottom, and to fill voids below the pipe.

X1.6.2 *Embedment Compaction*

Embedment compaction requirements should be determined by the engineer based on deflection limits established for the pipe, pipe stiffness, and installation quality control, as well as the characteristics of the in-situ soil and compactibility characteristics of the embedment materials used. The compaction requirements given in Table 3 are based on attaining an average modulus of soil reaction (E′) of 1000 psi[5] (or an appropriate equivalent constrained modulus, M_s), which relates soil stiffness to soil type and degree of compaction. For particular installations, the project engineer should verify that the percent compaction specified meets performance requirements.

X1.7 *Compaction Methods*—Achieving desired compaction for specific types of materials depends on the methods used to impart compactive energy. Coarse-grained, clean materials such as crushed stone, gravels, and sand are more readily compacted using vibratory equipment, whereas fine materials with high plasticity require kneading and impact force along with controlled water content to achieve acceptable compaction (see 5.3). In pipe trenches, small, hand-held or walk-behind compactors are required, not only to preclude damage to the pipe, but to ensure thorough compaction in the confined areas around the pipe and along the trench wall. As examples, vibratory plate tampers work well for coarse grained materials of Class I and Class II, whereas hand tampers or air driven hand-held impact rammers are suitable for the fine-grained, plastic groups of Class III and IV. Gas or diesel powered jumping jacks or small, walk-behind vibratory rollers impart both vibratory and kneading or impact force, and hence are suitable for most classes of embedment and backfill material.

X1.8 *Migration*—When coarse and open-graded material is placed adjacent to a finer material, fines may migrate into the coarser material under the action of hydraulic gradient from ground water flow. Significant hydraulic gradients may arise in the pipeline trench during construction when water levels are being controlled by various pumping or well-pointing methods, or after construction when permeable underdrain or embedment materials act as a "french'' drain under high ground water levels. Field experience shows that migration can result in significant loss of pipe support and continuing deflections that may exceed design limits. The gradation and relative size of the embedment and adjacent materials must be compatible in order to minimize migration (see X1.8.1 below). In general, where significant ground water flow is anticipated, avoid placing coarse, open-graded Class I materials above, below, or adjacent to finer materials, unless methods are employed to impede

[5] Howard, Amster, "Modulus of Soil Reaction Values for Buried Flexible Pipe," *Journal of Geotechnical Engineering*, ASCE, Vol. 103, No. GT1, 1977.

migration such as the use of an appropriate stone filter or filter fabric along the boundary of the incompatible materials. To guard against loss of pipe support from lateral migration of fines from the trench wall into open-graded embedment materials, it is sufficient to follow the minimum embedment width guidelines in X1.10.

X1.8.1 The following filter gradation criteria may be used to restrict migration of fines into the voids of coarser material under a hydraulic gradient:

X1.8.1.1 $D_{15} / d_{85} < 5$ where D_{15} is the sieve opening size passing 15 % by weight of the coarser material and d_{85} is the sieve opening size passing 85 % by weight of the finer material, and

X1.8.1.2 $D_{50}/d_{50} < 25$ where D_{50} is the sieve opening size passing 50 % by weight of the coarser material and d_{50} is the sieve opening size passing 50 % by weight of the finer material. This criterion need not apply if the coarser material is well-graded (see Test Method D2487).

X1.8.1.3 If the finer material is a fine-grained soil (CL, CH, ML, or MH), then the following criterion may be used in lieu of X1.8.1.1: $D_{15} < 0.02$ in. (0.5 mm) where D_{15} is the sieve opening size passing 15 % by weight of the coarser material.

NOTE X1.3—Materials selected for use based on filter gradation criteria, such as in X1.8.1, should be handled and placed in a manner that will minimize segregation.

X1.9 *Maximum Particle Size*—Limiting particle size to ¾ in. (20 mm) or less enhances placement of embedment material for nominal pipe sizes 8 in. (200 mm) through 15 in. (380 mm). For smaller pipe, a particle size of about 10 % of the nominal pipe diameter is recommended.

X1.10 *Embedment Width for Adequate Support*—In certain conditions, a minimum width of embedment material is required to ensure that adequate embedment stiffness is developed to support the pipe. These conditions arise where in-situ lateral soil resistance is negligible, such as in very poor native soils or along highway embankments. Examples of poor native soils include poorly compacted soils with blow counts of five or less, peat, muck, or highly expansive soils. Under these conditions, if the native soil is able to sustain a vertical cut, the minimum embedment width shall be 0.5 pipe diameters on either side of the pipe as shown in Fig. X1.1. Under these conditions, if the native soil cannot sustain a vertical cut or if it is an embankment situation, the minimum embedment width shall be one pipe diameter on either side of the pipe as shown in Fig. X1.2. In either case, the embedment material shall be a Class II granular material or a Class I crushed rock as specified in Section 5 of this standard. If other embedment materials are used, the engineer should establish the minimum embedment width based on an evaluation of parameters such as pipe stiffness, embedment stiffness, nature of in-situ soil, and magnitude of construction and service loads. Regardless of the trench width required for adequate support, the trench must be of sufficient width to allow the proper placement of embedment in accordance with 6.3.

NOTE X1.4—Installation in very poor soil conditions may require additional treatment, for example, soil stabilization or permanent sheeting.

NOTE X1.5—The embedment over the top of the pipe shown in Fig. X1.1 and Fig. X1.2 represent minimum cover for impact protection, not for pipe support. Regardless of the minimum cover shown, the requirements of 7.6 must be met.

NOTE X1.6—Refer to X1.6 for backfill material and compaction requirements to control deflection.

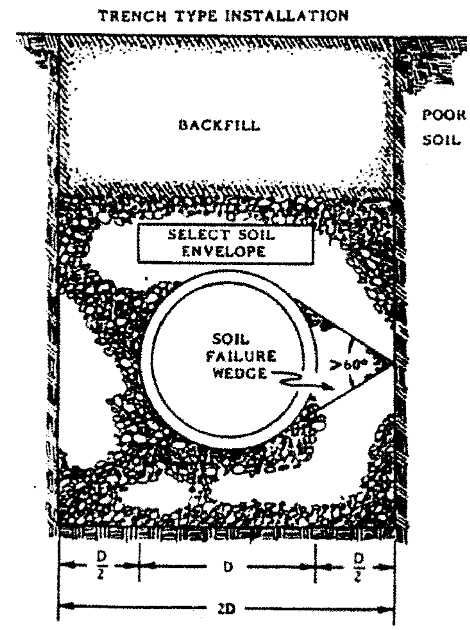


FIG. X1.1 Minimum Embedment Width When Trench and Native Soil Can Sustain a Vertical Cut

X1.11 *Lumps, Clods and Boulders*—Backfill materials should be free of lumps, clods, boulders, frozen matter, and debris. The presence of such material in the embedment may preclude uniform compaction and result in excessive localized deflections.

X1.12 *Other Design and Construction Criteria* —The design and construction of the pipe system should recognize conditions that may induce excessive shear, longitudinal bending, or compression loading in the pipe. Live loads applied by construction and service traffic may result in large, cumulative pipe deflections if the pipe is installed with a low density embedment and shallow cover. Other sources of loads on buried pipes are: freezing and thawing of the ground in the vicinity of the pipe, rising and falling of the ground water table, hydrostatic pressure due to ground water, and localized differential settlement loads occurring next to structures such as manholes and foundations. Where external loads are deemed to be excessive, the pipe should be installed in casing pipe or other load limiting structures.

X1.13 *Deflection Testing*—To ensure specified deflection limits are not exceeded, the engineer may require deflection testing of the pipe using specified measuring devices. To allow for stabilization of the pipe soil system, deflection tests should be performed at least 30 days after installation. However, as a quality control measure, periodic checks of deflection may be made during installation.

OPEN DITCH OR FILL TYPE INSTALLATION

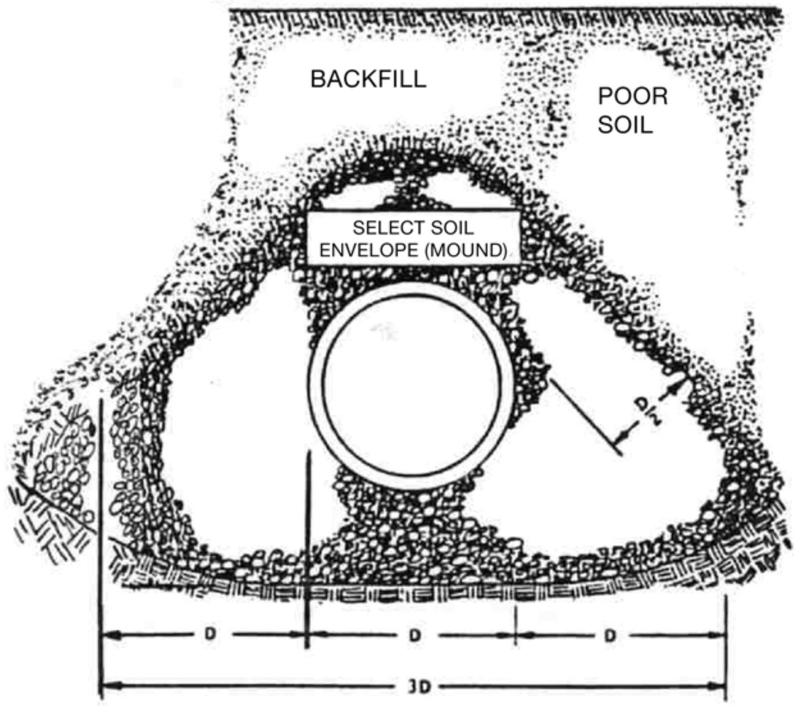


FIG. X1.2 Minimum Embedment Width When Native Soil Can Not Sustain a Vertical Cut or When Installed in the Embankment Condition

X1.13.1 Optional devices for deflection testing include electronic deflectometers, calibrated television or video cameras, or a properly sized "go, no-go'' mandrel. Deflection measurements can be made directly with extension rulers or tape measures in lines that permit safe entry. To ensure accurate measurements, clean the lines before testing.

X1.14 *Additional Installation Information*–Supplemental information useful for buried pipe installation can be found in Practice F1668.

X2. RECOMMENDATIONS FOR INCORPORATION IN CONTRACT DOCUMENTS

X2.1 This practice may be incorporated, by referral, into contract documents for a specific project to cover requirements for installation of flexible thermoplastic pipe in sewers and other gravity-flow applications. Application to a particular project should be made by means of a list of supplemental requirements. Suggested modifications to specific sections are listed below (the list is keyed to applicable section numbers of this practice):

X2.2 *Sections 5.1, 5.2, and Table 3* —Further restrictions on use of Classes of embedment and backfill materials.

X2.3 *Section 5*—Specific gradations of embedment materials for resistance to migration.

X2.4 *Section 5.5*—Maximum particle size, if different from Table 2.

X2.5 *Section 6.2*—Restrictions on mode of dewatering; design of underdrains.

X2.6 *Section 6.3*—Requirements on minimum trench width.

X2.7 *Section 6.4*—Restrictions or details for support of trench walls.

X2.8 *Section 7.5*—Specific restrictions on methods of compaction.

X2.9 *Section 7.5.1 and Table 3* —Minimum embedment percent compaction if different from these recommendations; specific compaction requirements for backfill (for example, for pavement subgrade).

X2.10 *Section 7.6*—Minimum cover requirements if different from this paragraph.

X2.11 *Section 7.7*—Detailed requirements for support of vertical risers, standpipes, and stacks to accommodate anticipated relative movements between pipe and such appurtenances. Detailing to accommodate thermal movements, particularly at risers.

X2.12 *Section 7.10*—Detailed requirements for manhole connections.

X2.13 *Section 7.11*—Requirements on methods of testing compaction and leakage.

X2.14 *Section X1.13*—Requirements on deflection and deflection measurements, including method and time of testing.

SUMMARY OF CHANGES

Committee F17 has identified the location of selected changes to this standard since the last issue (D2321–09) that may impact the use of this standard. (Approved Feb. 1, 2011.)

(*1*) 7.4.1 was revised to add gasket precautions and to eliminate "homing".

Committee F17 has identified the location of selected changes to this standard since the last issue (D2321–08) that may impact the use of this standard. (Approved Dec. 15, 2009)

(*1*) 2.1 and X1.14 – Added reference to Specification F1668.
(*2*) Section 3 – Added and deleted definitions consistent with other changes, including terms from Terminology D653.
(*3*) 7.5.1 – Revised wording in terms of "percent compaction;" added reference to constrained modulus, M_s.
(*4*) Fig. 1 - Changed height of initial backfill over pipe to "minimum 6 in (150 mm);" re-defined haunching zone.
(*5*) Table 2 – Corrected percent of fines for Class III and Class IV soils; added Note F.
(*6*) Table 3 – Modified "General Recommendations and Restrictions" for Class II fine sands (SP); modified "Embedment Compaction" requirements for GW and GP soils; modified "Foundation" requirements for Class IV soils.
(*7*) X1.4.1 – Removed reference to D3839 regarding construction deflection allowances.
(*8*) X1.4.4 – Removed incorrect definition of deflection lag factor.
(*9*) X1.6.2 – Added reference to constrained modulus, M_s.
(*10*) X1.8.1 – Clarified that both X1.8.1.1 and X1.8.1.2 are necessary migration criteria.
(*11*) X1.8.1.3 – Expanded the soil groups that fall within this alternate criterion for migration.
(*12*) Note X1.4 – Changed "hydraulic or under consolidated soils" to "very poor soil conditions."
(*13*) Entire standard – Revised wording for "density" and "Proctor" to "percent compaction."

Designation: D 3839 – 02$^{\epsilon 1}$

An American National Standard

Standard Guide for Underground Installation of "Fiberglass" (Glass-FiberReinforced Thermosetting-Resin) Pipe[1]

This standard is issued under the fixed designation D 3839; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This standard has been approved for use by agencies of the Department of Defense.

ϵ^1 NOTE—Table 3 was editorially revised in November 2003.

1. Scope

1.1 This practice establishes procedures for the burial of pressure and nonpressure "fiberglass" (glass-fiber-reinforced thermosetting-resin) pipe in many typically encountered soil conditions. Included are recommendations for trenching, placing pipe, joining pipe, placing and compacting backfill, and monitoring deflection levels. Guidance for installation of fiberglass pipe in subaqueous conditions is not included.

1.2 Product standards for fiberglass pipe encompass a wide range of product variables. Diameters range from 1 in. to 12 ft (25 mm to 3600 mm) and pipe stiffness range from 9 to over 72 psi (60 to 500 kPa) with internal pressure ratings up to several thousand pound-force per square inch. This standard does not purport to consider all of the possible combinations of pipe, soil types, and natural ground conditions that may occur. The recommendations in this practice may need to be modified or expanded to meet the needs of some installation conditions. In particular, fiberglass pipe with diameters of a few inches are generally so stiff that they are frequently installed in accordance with different guidelines. Consult with the pipe manufacturer for guidance on which practices are applicable to these particular pipes.

1.3 The scope of this practice excludes product-performance criteria such as a minimum pipe stiffness, maximum service deflection, or long-term strength. Such parameters may be contained in product standards or design specifications, or both, for fiberglass pipe. It is incumbent upon the specified product manufacturer or project engineer to verify and ensure that the pipe specified for an intended application, when installed in accordance with procedures outlined in this practice, will provide a long-term, satisfactory performance in accordance with criteria established for that application.

NOTE 1—There is no similar or equivalent ISO standard.

NOTE 2—A discussion of the importance of deflection and a presentation of a simplified method to approximate field deflections are given in AWWA Manual of Practice M45 Fiberglass Pipe Design.

1.4 *This standard does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*

D 8 Terminology Relating to Materials for Roads and Pavements[2]

D 653 Terminology Relating to Soil, Rock, and Contained Fluids[3]

D 698 Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft (600 kN-m/m))[3]

D 883 Terminology Relating to Plastics[4]

D 1556 Test Method for Density and Unit Weight of Soil in Place by the Sand-Cone Method[3]

D 1557 Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort (56 000 ft-lbf/ft (2 700 kN-m/m))[3]

D 2167 Test Method for Density and Unit Weight of Soil in Place by the Rubber Balloon Method[3]

D 2216 Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock[3]

D 2321 Practice for Underground Installation of Flexible Thermoplastic Pipe for Sewers and Other Gravity-Flow Applications[5]

D 2487 Classification of Soils for Engineering Purposes[5]

D 2488 Practice for Description of Soils (Visual-Manual Procedure)[3]

[1] This practice is under the jurisdiction of ASTM Committee D20 on Plastics and is the direct responsibility of Subcommittee D20.23 on Reinforced Plastic Piping Systems and Chemical Equipment.

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[2] *Annual Book of ASTM Standards*, Vol 04.03.

[3] *Annual Book of ASTM Standards*, Vol 04.08.

[4] *Annual Book of ASTM Standards*, Vol 08.01.

[5] *Annual Book of ASTM Standards*, Vol 08.04.

D 2922 Test Methods for Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth)[3]
D 3017 Test Method for Moisture Content of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth)[3]
D 4253 Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table[3]
D 4254 Test Method for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density[3]
D 4318 Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils[3]
D 4564 Test Method for Density of Soil in Place by the Sleeve Method[3]
D 4643 Method for Determination of Water (Moisture) Content of Soil by the Microwave Oven Method[3]
D 4914 Test Method for Density of Soil and Rock in Place by the Sand Replacement Method in a Test Pit[3]
D 4944 Test Method for Field Determination of Water (Moisture) Content of Soil by the Calcium Carbide Gas Pressure Tester Method[6]
D 4959 Test Method for Determination of Water (Moisture) Content of Soil by Direct Heating Method[6]
D 5030 Test Methods for Density and Unit Weight of Soil and Rock in Place by the Water Replacement Method in a Test Pit[6]
D 5080 Test Method for Rapid Determination of Percent Compaction[6]
F 412 Terminology Relating to Plastic Piping Systems[5]

2.2 *Other Standards:*

AASHTO LRFD Bridge Design Specifications, 2nd Edition, American Association of State Highway and Transportation Officials[7]
AAHSTO M145 Classification of Soils and Soil Aggregate Mixtures[7]
AWWA C 950 American Water Works Association Standard Specification for Fiberglass Pressure Pipe[8]
AWWA Manual of Practice M45 Fiberglass Pipe Design Manual[8]

3. Terminology

3.1 *Definitions:*

3.1.1 *General*—Unless otherwise indicated, definitions are in accordance with Terminologies D 8, D 653, D 883, and F 412.

3.2 *Definitions of Terms Specific to This Standard:* Descriptions of Terms Specific to This Standard:

3.2.1 *bedding*—backfill material placed in the bottom of the trench or on the foundation to provide a uniform material on which to lay the pipe.

3.2.2 *deflection*—any change in the inside diameter of the pipe resulting from installation or imposed loads, or both; deflection may be either vertical or horizontal and is usually reported as a percentage of the nominal inside pipe diameter.

3.2.3 *engineer*—the engineer in responsible charge of the work or his duly recognized or authorized representative.

3.2.4 *fiberglass pipe*—a tubular product containing glass-fiber reinforcements embedded in or surrounded by cured thermosetting resin; the composite structure may contain aggregate, granular, or platelet fillers, thixotropic agents, pigments, or dyes; thermoplastic or thermosetting liners or coatings may be included.

3.2.5 *final backfill*—backfill material placed from the top of the initial backfill to the ground surface.

3.2.6 *fines*—soil particles that pass a No. 200 seive.

3.2.7 *foundation*—in situ soil or, in the case of unsuitable ground conditions compacted backfill material, in the bottom of the trench the supports the bedding and the pipe (see Fig. 1).

3.2.8 *geotextile*—any permeable textile material used with foundation, soil, earth, rock, or any other geotechnical engineering related material, as an integral part of a man-made product, structure, or system.

3.2.9 *haunching*—backfill material placed on top of the bedding and under the springline of the pipe; the term haunching only pertains to soil directly beneath the pipe (see Fig. 1).

3.2.10 *initial backfill*—backfill material placed at the sides of the pipe and up to 6 to 12 in. (150 to 300 mm) over the top of the pipe, including the haunching.

3.2.11 *manufactured aggregates*—aggregates that are products or by-products of a manufacturing process, or natural aggregates that are reduced to their final form by a manufacturing process such as crushing.

3.2.12 *maximum standard Proctor density*—the maximum dry unit weight of soil compacted at optimum moisture content, as obtained by laboratory test in accordance with Test Method D 698.

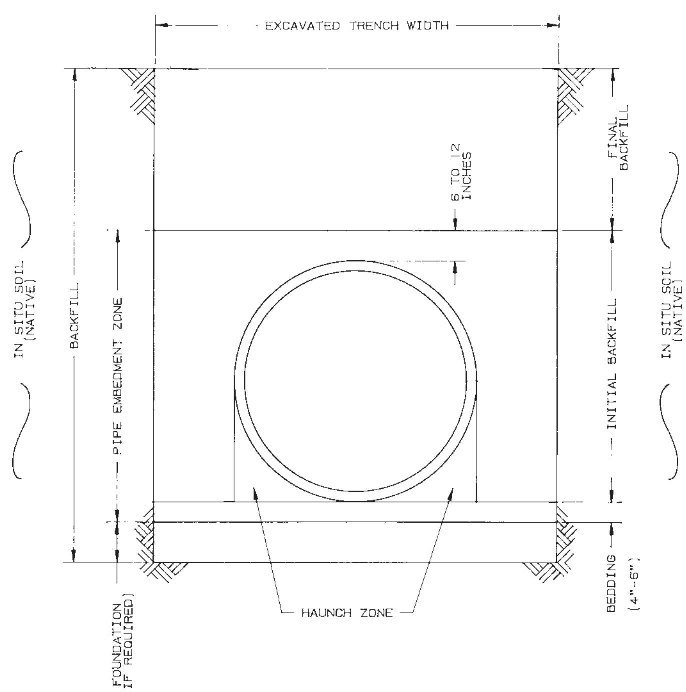

FIG. 1 Trench Cross-Section Terminology

[6] *Annual Book of ASTM Standards*, Vol 04.09.

[7] Available from American Association of State Highway and Transportation Officials (AASHTO), 444 N. Capitol St., NW, Suite 249, Washington, DC 20001.

[8] Available from American Water Works Association (AWWA), 6666 West Quincy Ave., Denver CO 80235.

3.2.13 *native (in situ) soil*—natural soil in which a trench is excavated for pipe installation or on which a pipe and embankment are placed.

3.2.14 *open-graded aggregate*—an aggregate that has a particle-size distribution such that, when compacted, the resulting voids between the aggregate particles, expressed as a percentage of the total space occupied by the material, are relatively large.

3.2.15 *optimum moisture content*—the moisture content of soil at which its maximum density is obtained. (See Test Method D 698.)

3.2.16 *pipe zone embedment*—all backfill around the pipe; this includes the bedding, haunching, and initial backfill.

3.2.17 *processed aggregates*—aggregates which are screened or washed or mixed or blended to produce a specific particle-size distribution.

3.2.18 *relative density*—a measure of the density of a granular soil based on the actual density of the soil "relative" to the soil in its loosest state and the soil in its densest state (see Terminology D 653 for a precise definition) as obtained by laboratory testing in accordance with Test Methods D 4253 and D 4254.

3.2.19 *soil stiffness*—a property of soil, generally represented numerically by a modulus of deformation that indicates the relative amount of deformation that will occur under a given load.

3.2.20 *split installation*—an installation in which the initial backfill consists of two different materials; the first material extends from the top of the bedding to a depth of at least 0.6 times the diameter and the second material extends to the top of the initial backfill.

4. Significance and Use

4.1 This practice is for use by designers and specifiers, manufacturers, installation contractors, regulatory agencies, owners, and inspection organizations involved in the construction of buried fiberglass pipelines. As with any practice, modifications may be required for specific job conditions, or for special local or regional conditions. Recommendations for inclusion of this practice in contract documents for a specific project are given in Appendix X1.

5. Materials

5.1 *Classification*—Soil types used or encountered in burying pipes include those classified in Table 1 and natural, manufactured, and processed aggregates. The soil classifications are grouped into soil-stiffness categories (SC#) in Table 2 based on the typical soil stiffness when compacted. Category SC1 indicates a soil that generally provides the highest soil stiffness at any given percentage of maximum Proctor density, and a soil that provides a given soil stiffness with the least compactive effort. Each higher-number soil-stiffness category provides successively less soil stiffness at a given percentage of maximum Proctor density and requires greater compactive effort to provide a given level of soil stiffness.

NOTE 3—See Practices D 2487 and D 2488 for laboratory and field visual-manual procedures for identification of soils.

NOTE 4—Processed materials produced for highway construction, including coarse aggregate, base, subbase, and surface coarse materials, when used for foundation, embedment, and backfill, should be categorized in accordance with this section and Table 1 in accordance with particle size and gradation.

5.2 *Installation and Use*—Table 3 provides recommendations on installation and use based on soil-stiffness category and location in the trench. Categories SC1 to SC4 should be used as recommended in Table 3. Soil-stiffness Category 5, including clays and silts with liquid limits greater than 50, organic soils, and frozen soils, shall be excluded from the pipe-zone embedment.

5.2.1 *Soil-Stiffness Category 1 (SC1)*—SC1 materials provide maximum stability and pipe support for a given percent compaction due to the low content of sand and fines. With minimum effort these materials can be installed at relatively high-soil stiffnesses over a wide range of moisture contents. In addition, the high permeability of SC1 materials may aid in the control of water, and these materials are often desirable for embedment in rock cuts where water is frequently encountered. However, when ground-water flow is anticipated, consideration should be given to the potential for migration of fines from adjacent materials into the open-graded SC1 materials. (See 5.5.)

5.2.2 *Soil-Stiffness Category 2 (SC2)*—SC2 materials, when compacted, provide a relatively high level of pipe support; however, open-graded groups may allow migration and the sizes should be checked for compatibility with adjacent material; see 6.5.

5.2.3 *Soil-Stiffness Category 3 (SC3)*—SC3 materials provide less support for a given density than SC1 or SC2 materials. Higher levels of compactive effort are required and moisture content must be near optimum to minimize compactive effort and achieve the required density. These materials provide reasonable levels of pipe support once proper density is achieved.

5.2.4 *Soil-Stiffness Category 4 (SC4)*—SC4 materials require a geotechnical evaluation prior to use. Moisture content must be near optimum to minimize compactive effort and achieve the required density. Properly placed and compacted, SC4 materials can provide reasonable levels of pipe support; however, these materials may not be suitable under high fills, surface-applied wheel loads, or under high-energy-level vibratory compactors and tampers. Do not use where water conditions in the trench may prevent proper placement and compaction.

NOTE 5—The term "high energy level vibratory compactors and tampers" refers to compaction equipment that might deflect or distort the pipe more than permitted by the specifications or the manufacturer.

5.2.5 *Soil-Stiffness Category 5 (SC5)*— SC5 materials should be excluded from pipe-zone embedment.

5.3 *Moisture Content of Embedment Materials*—The moisture content of embedment materials must be controlled to permit placement and compaction to required levels. For non-free draining soils (that is, SC3 and SC4 and some borderline SC2 soils), moisture content is normally controlled to ±3 % of optimum (see Test Method D 698). The practicality of obtaining and maintaining the required limits on moisture content is an important criterion for selecting materials, since

TABLE 1 Soil Classification Chart (see Classification D 2487)

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests[A]				Soil Classification	
				Group Symbol	Group Name[B]
Coarse-Grained Soils More than 50 % retained on No. 200 sieve	gravels more than 50 % of coarse fraction retained on No. 4 sieve	clean gravels less than 5 % fines[E]	Cu ≥ 4 and 1 ≤ Cc ≤ 3[C]	GW	well-graded gravel[D]
			Cu < 4 and/or 1 > Cc > 3[C]	GP	poorly graded gravel[D]
		gravels with fines more than 12 % fines[E]	Fines classify as ML or MH	GM	silty gravel[D,F,G]
			Fines classify as CL or CH	GC	clayey gravel[D,F,G]
	sands 50 % or more of coarse fraction passes No. 4 sieve	clean sands less than 5 % fines[I]	Cu ≥ 6 and 1 ≤ Cc ≤ 3[C]	SW	well-graded sand[H]
			Cu < 6 and/or 1 > Cc > 3[C]	SP	poorly graded sand[H]
		sands with fines more than 12 % fines[I]	Fines classify as ML or MH	SM	silty sand[F,G,H]
			Fines classify as CL or CH	SC	clayey sand[F,G,H]
Fine-Grained Soils 50 % or more passes the No. 200 sieve	silts and clays liquid limit less than 50	inorganic	PI > 7 and plots on or above "A" line[J]	CL	lean clay[K,L,M]
			PI < 4 or plots below "A" line[J]	ML	silt[K,L,M]
		organic	$\frac{\text{liquid limit} - \text{oven dried}}{\text{liquid limit} - \text{not dried}} < 0.75$	OL	organic clay[K,L,M,N] / organic silt[K,L,M,O]
	silts and clays liquid limit 50 or more	inorganic	PI plots on or above "A" line	CH	fat clay[K,L,M]
			PI plots below "A" line	MH	elastic silt[K,L,M]
		organic	$\frac{\text{liquid limit} - \text{oven dried}}{\text{liquid limit} - \text{not dried}} < 0.75$	OH	organic clay[K,L,M,P] / organic silt[K,L,M,Q]
Highly organic soils	primarily organic matter, dark in color, and organic odor			PT	peat

[A] Based on the material passing the 3-in. (75-mm) sieve.
[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.
[C] $Cu = D_{60}/D_{10}$ $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$
[D] If soil contains ≥15 % sand, add "with sand" to group name.
[E] Gravels with 5 to 12 % fines require dual symbols:
GW-GM well-graded gravel with silt
GW-GC well-graded gravel with clay
GP-GM poorly graded gravel with silt
GP-GC poorly graded gravel with clay
[F] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.
[G] If fines are organic, add "with organic fines" to group name.
[H] If soil contains ≥15 % gravel, add "with gravel" to group name.
[I] Sands with 5 to 12 % fines require dual symbols:
SW-SM well-graded sand with silt
SW-SC well-graded sand with clay
SP-SM poorly graded sand with silt
SP-SC poorly graded sand with clay
[J] If Atterberg limits plot in hatched area, soil is a CL-ML, silty clay (see Test Method D 4318).
[K] If soil contains 15 to 29 % plus No. 200, add "with sand" or "with gravel," whichever is predominant.
[L] If soil contains ≥30 % plus No. 200, predominantly sand, add "sandy" to group name.
[M] If soil contains ≥30 % plus No. 200, predominantly gravel, add "gravelly" to group name.
[N] PI ≥ 4 and plots on or above "A" line.
[O] PI < 4 or plots below "A" line.
[P] PI plots on or above "A" line.
[Q] PI plots below "A" line.

failure to achieve required density, especially in the pipe zone embedment, may result in excessive deflection.

5.4 *Maximum Particle Size*—Maximum particle size for pipe-zone embedment is limited based on pipe diameter as listed in Table 4. For final backfill, the maximum particle size allowed should not exceed 75 % of the lift thickness. When final backfill contains cobbles, boulders, etc., the initial bedding should be extended above the top of the pipe at least 12 in. (300 mm). Backfill containing particles larger than 8 in. (200 mm) shall not be dropped on the backfill or rolled down a sloping trench wall from a height greater than 6 ft (1.8 m) until the depth of fill over the top of the pipe is greater than 24 in. (600 mm).

NOTE 6—The limits of 200 mm (8 in.) particles and a drop height of 6 ft (1.8 m) are somewhat arbitrary, but serve to establish the principle that dropping boulders onto the backfill can damage the pipe even though some backfill has already been placed on the pipe.

5.5 *Migration*—When open-graded material is placed adjacent to a finer material, fines may migrate into the coarser material under the action of hydraulic gradient from ground water flow. Significant hydraulic gradients may arise in the pipeline trench during construction, when water levels are being controlled by various pumping or well-pointing methods, or after construction, when permeable underdrain or embedment materials act as a "french" drain under high ground water levels. Field experience shows that migration can result in significant loss of pipe support and increasing deflections that may eventually exceed design limits. The gradation and relative size of the embedment and adjacent materials must be compatible in order to minimize migration. In general, where significant ground water is anticipated, avoid placing coarse, open-graded materials, such as SC1, above, below, or adjacent

TABLE 2 Soil-Stiffness Categories

NOTE 1—Soil stiffness categories group types together as a function of the relative level of soil stiffness developed when compacted to a given level. At any given level of compaction, SC1 soils provide the highest stiffness and SC5 soils the lowest.

NOTE 2—The soil-stiffness categories are similar but not identical to the soil classes in Practice D 2321.

Soil Group	Soil Stiffness Category
Crushed rock: 15 % sand, maximum 25 % passing the ⅜ in. sieve and maximum 5 % passing a #200 sieve	SC1
Clean, coarse grained soils: SW, SP, GW, GP or any soil beginning with one of these symbols with 12 % or less passing a #200 sieve	SC2
Coarse grained soils with fines: GM, GC, SM, SC, or any soil beginning with one of these symbols, containing more than 12 % passing a #200 sieve; Sandy or gravelly fine-grained soils: CL, ML, (or CL-ML, CL/ML. ML/CL) with more than 30 % retained on a #200 sieve	SC3
Fine-grained soils: CL, ML, (or CL-ML, CL/ML. ML/CL) with 30 % or less retained on a #200 sieve	SC4
MH, CH, OL, OH, PT	SC5 Not for use as embedment

to finer materials, unless methods are employed to impede migration such as the use of an appropriate soil filter or a geotextile filter fabric along the boundary of the incompatible materials.

5.5.1 The following filter gradation criteria may be used to restrict migration of fines into the voids of coarser material under a hydraulic gradient:

$$D_{15}/d_{85} < 5 \quad (1)$$

where:

D_{15} = sieve opening size passing 15 % by weight of the coarser material, and
d_{85} = sieve opening size passing 85 % by weight of the finer material.

$$D_{50}/d_{50} < 25 \quad (2)$$

where:

D_{50} = sieve opening size passing 50 % by weight of the coarser material, and
d_{50} = sieve opening size passing 50 % by weight of the finer material. This criterion need not apply if the coarser material is well-graded (see Classification D 2487).

5.5.2 If the finer material is a medium to highly plastic clay without sand particles (CL or CH), then the following criterion may be used instead of 6.5.1:

$$D_{15} < 0.02\ in.\ (0.5\ mm) \quad (3)$$

where:

D_{15} = sieve-opening size passing 15 % by weight of the coarser material.

NOTE 7—Materials selected for use based on filter-gradation criteria such as in 6.5 should be handled and placed in a manner that will minimize segregation.

5.6 *Cementitious Backfill Materials*—Backfill materials supplemented with cement to improve long-term strength and/or stiffness (soil cement, cement stabilized backfill) or to improve flowability (flowable fill, controlled low strength material) have been shown to be effective backfill materials in terms of ease of placement and quality of support to pipe. While not specifically addressed by this standard, use of these materials is beneficial under many circumstances.

6. Trench Excavation

6.1 *Excavation*—Excavate trenches to ensure that sides will be stable under all working conditions. Slope trench walls or provide supports in conformance with all local and national standards for safety. Place excavated material away from the edge of the trench. Open only enough trench that can be safely maintained by available equipment. Place and compact backfill in trenches as soon as practicable, preferably no later than the end of each working day.

6.2 *Water Control*—It is always good practice to remove water from a trench before laying and backfilling pipe. While circumstances occasionally require pipe installation in standing or running water conditions, such practice is outside the scope of this practice. At all times prevent run-off and surface water from entering the trench.

6.2.1 *Ground Water*—When ground water is present in the work area, dewater to maintain stability of in situ and imported materials. Maintain the water level below pipe bedding. Use, as appropriate, sump pumps, well points, deep wells, geotextiles, perforated underdrains or stone blankets of sufficient thickness to remove and control water in the trench. When excavating while lowering the ground water level, ensure that the ground water is below the bottom of cut at all times to prevent washout from behind sheeting or sloughing of exposed trench walls. Maintain control of water in the trench before, during, and after pipe installation, and until embedment is installed and sufficient backfill has been placed to prevent flotation of the pipe. To preclude loss of soil support, employ dewatering methods that minimize removal of fines and the creation of voids in in situ materials.

6.2.2 *Running Water*—Control running water emanating from surface drainage or ground water to preclude undermining of the trench bottom or walls, the foundation, or other zones of embedment. Provide dams, cutoffs, or other barriers periodically along the installation to preclude transport of water along the trench bottom. Backfill all trenches as soon as practical after the pipe is installed to prevent disturbance of pipe and embedment.

6.2.3 *Materials for Water Control*—Use suitably graded materials in the foundation as drainage blankets for transport of running water to sump pits or other drains. Use properly graded materials or perforated underdrains, or both, to enhance transport of running water. Select the gradation of the drainage materials to minimize migration of fines from surrounding materials. (See 5.5.)

6.3 *Minimum Trench Width*—Where trench walls are stable or supported, provide a width sufficient, but no greater than necessary, to ensure working room to properly and safely place

TABLE 3 Recommendations for Installation and Use of Soils and Aggregates for Foundation and Pipe-Zone Embedment

Soil Stiffness Category[A]	SC1	SC2	SC3	SC4
General Recommendations and Restrictions	Acceptable and common where no migration is probable or when combined with a geotextile filter media. Suitable for use as a drainage blanket and under drain where adjacent material is suitably graded or when used with a geotextile filter fabric (see 6.5).	Where hydraulic gradient exists check gradation to minimize migration. Clean groups are suitable for use as a drainage blanket and underdrain (see Table 2). Uniform fine sands (SP) with more than 50 % passing a #100 sieve (0.006 in., 0.15 mm) behave like silts and should be treated as SC3 soils.	Do not use where water conditions in trench prevent proper placement and compaction. Not recommended for use with pipes with stiffness of 9 psi or less	Difficult to achieve high-soil stiffness. Do not use where water conditions in trench prevent proper placement and compaction. Not recommended for use with pipes with stiffness of 9 psi or less
Foundation	Suitable as foundation and for replacing over-excavated and unstable trench bottom as restricted above.	Suitable as foundation and for replacing over-excavated and unstable trench bottom as restricted above. Install and compact in 12 in. (300 mm) maximum layers	Suitable for replacing over-excavated trench bottom as restricted above. Install and compact in 6 in. (150 mm) maximum layers	Not suitable.
Pipe Zone Embedment	Suitable as restricted above. Work material under pipe to provide uniform haunch support.	Suitable as restricted above. Work material under pipe to provide uniform haunch support.	Suitable as restricted above. Difficult to place and compact in the haunch zone.	Suitable as restricted above. Difficult to place and compact in the haunch zone.
Embedment Compaction:				
Min Recommended Density, SPD[B]	[C]	85 %	90 %	95 %
Relative Compactive Effort Required to Achieve Minimum Density	low	moderate	high	very high
Compaction Methods	vibration or impact	vibration or impact	impact	impact
Required Moisture Control	none	none	maintain near optimum to minimize compactive effort	maintain near optimum to minimize compactive effort

[A] SC5 materials are unsuitable as embedment. They may be used as final backfill as permitted by the engineer.
[B] SPD is standard Proctor density as determined by Test Method D 698.
[C] Minimum density typically achieved by dumped placement.

TABLE 4 Maximum Particle Size for Pipe Embedment

Nominal Diameter (D_i)) Range, in. (mm)	Maximum Particle Size, in., (mm)
D_i= 18 (D_i= 450)	0.50, (13)
18< D_i= 24 (450< D_i= 600)	0.75 (19)
24< D_i= 36 (600< D_i= 900)	1.00 (25)
36< D_i= 48 (900< D_i= 1200)	1.25 (32)
48< D_i(1200< D_i)	1.50 (38)

and compact haunching and other embedment materials. The space between the pipe and trench wall must be wider than the compaction equipment used in this region. For a single pipe in a trench, the minimum trench width should be 1.25 times the outside diameter of the pipe plus 12 in. (300 mm). For multiple pipes in the same trench, interior spaces between pipes must be at least the average of the radii of the two adjacent pipe for depths greater than 12 ft (3.5 m), and ⅔ of the average of the radii of the two adjacent pipe for depths less than 12 ft (3.5 m); the distance from the outside pipe to the trench wall must not be less than if that pipe were installed as a single pipe in a trench. If mechanical compaction equipment is used, the minimum space between pipe and trench wall, or between adjacent pipe shall not be less than the width of the widest piece of equipment plus 6 in. (150 mm). In addition to safety considerations, trench width in unsupported, unstable soils will depend on the size and stiffness of the pipe, stiffness of the embedment and in situ soil, and depth of cover. Specially designed equipment may facilitate the satisfactory installation and embedment of pipe in trenches narrower than specified above. If it is determined that the use of such equipment provides an installation consistent with the requirements of this practice, minimum trench widths may be reduced if approved by the engineer.

6.4 *Support of Trench Walls*—When supports such as trench sheeting, trench jacks, trench shields, or boxes are used, ensure that support of the pipe and the embedment is maintained throughout the installation process. Ensure that sheeting is sufficiently tight to prevent washing out of the trench wall from behind the sheeting. Provide tight support of trench walls below viaducts, existing utilities, or other obstructions that restrict driving of sheeting.

6.4.1 *Support Left in Place*—Unless otherwise directed by the engineer, sheeting driven below the top of the pipe should be left in place to preclude loss of support of foundation and embedment materials. When the top of the sheeting is to be cut off, make the cut 1.5 ft (0.5 m) or more above the crown of the pipe. Leave rangers, walers, and bracers in place as required to support cutoff sheeting and the trench wall in the vicinity of the pipe. Timber sheeting to be left in place is considered a permanent structural member, and should be treated against biological degradation (for example, attack by insects or other biological forms), as necessary, and against decay if above ground water.

NOTE 8—Certain preservative and protective compounds may pose environmental hazards. Determination of acceptable compounds is outside the scope of this practice.

6.4.2 *Movable Trench-Wall Supports*—Do not disturb the installed pipe and its embedment when using movable trench boxes and shields. Movable supports should not be used below the top of the pipe embedment zone, unless approved methods are used for maintaining the integrity of embedment material. Before moving supports, place and compact embedment to sufficient depths to ensure protection of the pipe. As supports are moved, finish placing and compacting embedment, and ensure the direct compaction of embedment materials against the undisturbed native soil.

6.4.3 *Removal of Trench-Wall Support*— If the engineer permits the use of sheeting or other trench-wall supports that extend below the top of the pipe, ensure that neither pipe, foundation, nor embedment materials is disturbed by support removal. Fill voids left on removal of supports and compact all material to required densities.

6.5 *Trench-Bottom*—Excavate trenches to a minimum depth of 4 in. (100 mm) below the pipe. See Section 7 for guidance on installing foundation and bedding.

6.5.1 When ledge, rock, hardpan or other unyielding material, cobbles, rubble or debris, boulders, or stones larger than 1.5 in. (38 mm) are encountered in the trench bottom, excavate a minimum depth of 6 in. (150 mm) below the pipe bottom, or as directed by the engineer.

6.5.2 If the trench bottom is unstable or shows a "quick" tendency, overexcavate to depths directed by the engineer.

6.6 *Trenching on Slopes*—The angle at which slopes can become unstable depends on the quality of the soil. The risk of unstable conditions increases dramatically with slope angle. In general, pipes should not be installed on slopes greater than 15 degrees (a slope of 1 to 4) or in areas where slope instability is suspected, unless supporting conditions have been verified by a proper geotechnical investigation. Installing pipes above ground may be a preferred method for steep slopes as above ground structures such as pipe supports are more easily defined and, therefore, the quality of installation is easier to monitor and settlement easier to detect. Pipes may be installed on slopes greater than 15 degrees (a slope of 1 to 4) provided that:

6.6.1 Long term stability of the installation can be ensured with proper geotechnical design.

6.6.2 Pipes are backfilled with coarse-grained material (SC1) with high shear strength or the shear strength of the backfill is assured by other means. The backfill should be compacted to at least 90 % of maximum standard Proctor density (Test Method D 698).

6.6.3 Pipes should be installed in straight alignment (plus or minus 0.2 degrees) with minimum gap between pipe ends.

6.6.4 Absolute long term movement of the backfill in the axial direction of the pipe must be less than 0.75 in. (20 mm) to avoid joint separation.

6.6.5 The installation is properly drained to avoid washout of materials and ensure adequate soil shear strength.

6.6.6 Stability of individual pipes is monitored throughout the construction phase and the first stages of operation.

6.6.7 The manufacturer is consulted to determine if a special pipe design is required.

7. Installation

7.1 *General*—Recommendations for use of the various types of materials classified in Section 5 and Table 1 for the foundation and pipe zone embedment are given in Table 3.

NOTE 9—Installation of pipe in areas where significant settlement may be anticipated, such as in backfill adjacent to building foundations, and in sanitary landfills, or in other highly unstable soils, require special engineering and are outside the scope of this practice.

7.2 *Foundation/Bedding*—Install foundation and bedding as required by the engineer in accordance with conditions in the trench-bottom. Provide a firm, stable, and uniform bedding for the pipe barrel and any protruding features of its joint. Provide a minimum of 4 in. (100 mm) of bedding below the barrel and 3 in. (75 mm) below any other part of the pipe unless otherwise specified.

7.2.1 *Bedding Material*—Often the bedding material will need to be an imported material to provide the proper gradation and pipe support. The bedding material should be the same material as the initial backfill. Native-soil material can be used as a bedding material if it meets the requirements of the initial backfill. This determination must be made as the pipe installation progresses because native-soil conditions vary widely and may change suddenly along the length of a pipeline. It is increasingly common to leave the bedding uncompacted for a width of ⅓ of the pipe diameter centered directly under the pipe. This reduces concentrated loads on the invert of the pipe.

7.2.2 *Rock and Unyielding Materials*— When rock or unyielding material is present in the trench bottom, install a cushion of bedding, 6-in. (150-mm) minimum thickness, below the bottom of the pipe.

7.2.3 *Unstable Trench-Bottom*—Where the trench-bottom is overexcavated because of unstable or "quick" conditions, install a foundation of SC1, SC2, or larger materials. Complete the foundation with a suitably graded material where conditions may cause migration of fines and loss of pipe support. For severe conditions, the engineer may require a special foundation such as piles or sheeting capped with a concrete mat. Control of quick and unstable trench-bottom conditions may be accomplished with the use of geotextiles.

7.2.4 *Localized Loadings*—Minimize localized loadings and differential settlement wherever the pipe crosses other utilities or subsurface structures, or whenever there are special foundations such as concrete-capped piles or sheeting. Provide a 6-in. (150-mm) minimum cushion of bedding between the pipe and any such point of localized loading.

7.2.5 *Over-Excavation*—If the trench bottom is over-excavated below intended grade, fill the over-excavation with compatible foundation or bedding material and compact to a density not less than the minimum densities stated in Table 3.

7.2.6 *Sloughing*—If trench sidewalls slough off during any excavation or installation of pipe-zone embedment, remove all sloughed and loose material from the trench.

7.3 *Location and Alignment*—Place pipe and fittings in the trench with the invert conforming to the required elevations, slopes, and alignment. Provide bell holes in pipe bedding, no larger than necessary, in order to ensure uniform pipe support. Fill all voids under the bell by working in bedding material. In special cases where the pipe is to be installed to a curved alignment, maintain angular "joint deflection" (axial alignment) or pipe-bending radius within acceptable design limits, or both. Pipe should be laid on flat uniform material that is at the appropriate grade. Do not bring pipe to grade by the use of mounds of soil or other material at points along the length of the pipe. When pipe laying is interrupted, secure piping against movement and seal open ends to prevent the entrance of water, mud, or foreign material.

7.4 *Jointing*—Comply with manufacturer's recommendations for assembly of joint components, lubrication, and making of joints.

7.4.1 *Elastomeric Seal Joints*—Mark, or verify that pipe ends are marked, to indicate insertion stop position, and that pipe is inserted into pipe or fitting bells to this mark. Push spigot into bell using methods recommended by the manufacturer, keeping pipe true to line and grade. Protect the end of the pipe during assembly and do not use excessive force that may result in over-assembled joints or dislodged gaskets. If full entry is not achieved, disassemble and clean joint and reassemble. Use only lubricant supplied or recommended for use by the manufacturer. Do not exceed the manufacturer's recommendations for angular "deflection" (axial alignment).

7.4.2 *Adhesive-Bonded or Wrapped Joints, or Both*—When making adhesive-bonded or wrapped joints, or both, follow recommendations of the pipe manufacturer. Allow freshly made joints to set for the recommended time before moving, burying, or otherwise disturbing the pipe.

NOTE 10—Axial restraint of the joined sections may be required during curing to prevent thermal expansion or contraction which could cause damage to the joint.

7.4.3 *Angularly Deflected Joints*—Large radius bends in pipelines may be accomplished by rotating the alignment of adjacent lengths of pipe (that is, "angularly deflecting" the joint). The amount of angular deflection should not exceed the manufacturer's recommendations.

7.5 *Placing and Compacting Backfill Materials*—Place embedment materials by methods which will not disturb or damage the pipe. Work in and compact the haunching material in the area between the bedding and the underside of the pipe before placing and compacting the remainder of the pipe-zone embedment. Follow recommendations for compaction given in Table 3 and this section. Do not permit compaction equipment to contact and damage the pipe. Use compaction equipment and techniques that are compatible with materials used and location in the trench. See 8.6 for requirements for minimum cover.

7.5.1 *Minimum Density*—The minimum embedment density should be established by the engineer based on an evaluation of specific project conditions. Table 3 gives recommendation for minimum densities that are applicable to most typical projects. Higher densities than those recommended in Table 3 may be appropriate and occasionally lower densities than those recommended in Table 3 may be acceptable.

NOTE 11—The traditional measure of soil stiffness has been the modulus of soil reaction, E', that is commonly used to predict flexible pipe deflection. Recently AASHTO has changed this parameter to the constrained soil modulus, M_s. See Appendix X2 for additional details.

7.5.2 *Densification with Water*—Densification of cohesionless material with water (jetting or saturation with vibration) should only be used under controlled conditions when approved by the engineer. Achieving a suitable water content in the soil is crucial and is best determined by trial test areas. Trial test areas may also be useful in determining the size of internal vibrators required and the appropriate spacing of their insertion into the soil.

7.5.3 *Compaction of Soils Containing Few Fines (Soil Stiffness Categories SC1 and SC2 with Less Than 5 % Fines)*—If compaction is required, use surface plate vibrators, vibratory rollers, or internal vibrators. The compacted lift thickness should not exceed 12 in. (300 mm) when compacted with surface plate vibrators or vibratory rollers and should not exceed the length of the internal vibrator. Density determination should normally be in accordance with Test Methods D 4253 and D 4254 (relative density). In some cases, the density of SW or SP soils may be determined by Test Method D 698 (standard proctor) if the test results in a clearly defined compaction curve.

7.5.4 *Compaction of Soils Containing Some Fines (Soil Stiffness Category SC2 with 5 to 12 % Fines)*—These soils may behave as a soil containing few fines (see 7.5.3) or as a soil containing a significant amount of fines (see 7.5.5). The methods of compaction and density determination should be those methods (7.5.3 or 7.5.5) that result in the higher in-place density.

7.5.5 *Compaction of Soils Containing a Significant Amount of Fines (Soil Stiffness Categories SC3, SC4, and SC5 (CH and MH)*—These soils should be compacted with impact tampers or with sheepsfoot rollers. Density determination should be in accordance with Test Method D 698 (standard Proctor). The maximum density occurs at the optimum moisture content. Less effort is required to reach a given density when the moisture content is within two percentage points of the optimum moisture. A rapid method of determining the percent compaction and moisture variation is described in Test Method D 5080. For compaction levels of 90 % standard Proctor and higher, the compacted lift thickness should not exceed 6 in. (150 mm).

7.5.6 *Determination of the In-Place Density of Soils*—The in-place density of any in situ or fill soil may be determined in accordance with Test Method D 1556, D 2167, D 2922, D 4564, D 4914, or D 5030. The applicable test method will depend on the type of soil, moisture content of the soil, and the maximum particle size present in the soil. The moisture content of the soil may be determined in accordance with Test Method D 2216, D 3017, D 4643, D 4944, or D 4959. When using nuclear density-moisture gages (Test Methods D 2922 and D 3017), the gage should be site-calibrated in the proximity of the pipe and in the excavation unless otherwise indicated by the gage manufacturer.

7.6 *Backfill Around Angularly Deflected Pipe Joints*—When pipe joints are angularly rotated to accomplish large radii bends in pipelines that will operate at internal pressures of 15 psig (100 kPa) or greater, the backfill surrounding the joint should be compacted to at least 90 % of maximum standard Proctor density (or appropriate alternate standard for soils with few fines) for SC1 and SC2 materials, and 95 % of maximum standard Proctor density for SC3 and SC4 materials. Consult the manufacturer for minimum depths of burial and additional restraint that may be required when the angular deflection is vertical.

7.7 *Minimum Cover*—To preclude damage to the pipe and disturbance to pipe embedment, a minimum depth of backfill above the pipe should be maintained before allowing vehicles or heavy construction equipment to traverse the pipe trench. The minimum depth of cover for surface loads, should be established by the engineer based on an evaluation of specific project conditions, including the pipe-zone embedment material and density, the native-soil characteristics, pipe stiffness, pipe diameter, surface pavement, surface loads, and final backfill compaction. In the absence of an engineering evaluation, the following minimum cover requirements should be used.

7.7.1 For embedment materials installed to the minimum densities given in Table 3, provide cover (that is, depth of backfill above top of pipe) of at least 24 in. (0.6 m) for SC1 embedment and a cover of at least 36 in. (0.9 m) for SC2, SC3, or SC4 embedment, before allowing vehicles or construction equipment to traffic the trench surface. Provide at least 48 in. (1.2 m) of cover before using a hydrohammer for compaction unless approved by the engineer. Where construction loads may be excessive (for example, cranes, earth-moving equipment, or other vehicles where wheel loads exceed the AASHTO HS-20 loading) minimum cover shall be increased as determined by the engineer. A minimum of one pipe diameter of cover is suggested to prevent flotation of an empty pipe when full soil saturation to the surface exists.

7.8 *Connections and Appurtenant Structures*:

7.8.1 *Connections to Manholes and Rigid Structures*—When differential settlement can be expected, such as at the ends of casing pipe, when the pipe enters a manhole or at anchor block, provide a flexible system capable of accommodating the anticipated settlement. This may be accomplished by placing a joint as close as practically possible to the face of the structure and a second joint within one to two pipe diameters from the face of the structure. The short length of pipe, called a rocker pipe shall be installed in a straight alignment with the short pipe section coming out of the rigid structure. Multiple rocker pipes should not be used. Alternatively, attach the pipe to the rigid structure with a flexible boot capable of accommodating the anticipated differential movement. Extra care and caution must be taken to replace and properly compact backfill adjacent to any rigid structure. Construction of concrete structures will frequently require over-excavation for formwork, etc. This extra-excavated material must be restored to a density level compatible with surroundings to prevent excess deformation and or joint rotation adjacent the structure. In these areas, compact backfill to achieve the same soil density as specified for all pipe backfill, but not less than that required to achieve a soil constrained modulus (M_a) of at least 1000 psi (6.9 MPa). Other methods of accommodating the differential settlements may be acceptable if approved in advance.

NOTE 12—The use of cement stabilized or flowable backfills adjacent to large structures has been found to be effective in preventing excess deformation where diameters are larger than about 60 in. (1,500 mm).

7.8.2 *Vertical Risers*—Provide support for vertical risers as commonly found at service connections, cleanouts, and drop manholes to preclude vertical or lateral movement. Prevent the direct transfer of thrust due to surface loads and settlement, and ensure adequate support at points of connection to main lines.

7.9 *Exposing Pipe for Making Service-Line Connections*—When excavating for a service-line connection, excavate material from above the top of the existing pipe before removing material from the sides of the pipe. Materials and density of service-line embedment should conform to the specifications for the existing line, or with this practice, whichever is more stringent.

7.10 *Pipe Caps and Plugs*—Secure caps and plugs to the pipe to prevent movement and resulting leakage under test and service pressures.

7.11 *Parallel Piping Systems*—Compact the soil between the pipes in the same manner as the soil between the pipe and the trench wall, taking special care to compact the soil in the haunch zone.

8. Monitoring, Inspecting, and Testing

8.1 *Field Monitoring*—Compliance with pipe installation requirements, including trench depth, grade, water conditions, foundation, embedment and backfill materials, joints, density of materials in place, and safety should be monitored to assure conformance with accordance with contract documents.

8.2 *Deflection*—Monitor the deflection level in the pipe throughout the installation process for conformance to the requirements of the contract specifications and the manufacturer's recommendations. Conduct deflection measurements early in a project to verify that construction procedures are adequate. The deflection at the time of installation will be less than the long-term deflection due to time-dependent load increase. If necessary, also consider the effects of vertical ovalling during compaction. Deflection testing should be completed prior to undertaking pressure tests.

8.3 *Pressure Testing*—Most pressure pipelines are tested after installation to detect leaks, installation flaws, damaged pipes or other deficiencies (see Appendix X1). As a general rule, such tests should not be conducted using air pressure, unless special precautions, not within the scope of this practice, are used. Additional recommendations for conducting pressure tests include:

8.3.1 Required thrust restraints are properly installed (and sufficiently cured if applicable).

8.3.2 Backfilling should be completed. Some sections of the line may be left uncovered provided suitable lateral and longitudinal restraint is provided.

8.3.3 Pumps and valves are anchored.

8.3.4 Assure test caps and endplugs are properly installed and restrained as necessary.

8.3.5 Vent the pipeline while filling to allow all air to escape.

8.3.6 Pressurize the line slowly to avoid pressure surges.

8.3.7 In determining the test pressure remember that the lowest point on the line will have the highest pressure. If the test pressure gage is not installed at this location, then the pressure should be determined by calculation.

8.3.8 Assure that the test fluid temperature is stable during the test period (to avoid pressure changes due to thermal expansion or contraction that may be misinterpreted as leaks).

9. Inspection, Handling, and Storage

9.1 *Inspection*—Upon receipt, inspect each shipment of pipe and fittings for conformance to product specifications and contract documents, and check for damage. Reject or repair nonconforming or damaged pipe. Remove rejected pipe from the jobsite.

9.2 *Handling and Storage*—Proper handling and storage of the pipe is important to achieve a successful installation. Consult the manufacturer for recommendations and appropriate procedures.

10. Keywords

10.1 backfill; bedding; fiberglass pipe; haunching; soil stiffness; underground installation

APPENDIXES

(Nonmandatory Information)

X1. RECOMMENDATION FOR INCORPORATION IN CONTRACT DOCUMENTS

This practice may be incorporated by referral in contract documents for a specific project, to cover requirements for installation. Applications to a particular project should be made by means of a list of supplemental requirements. Suggested modifications to specific section numbers are listed as follows. (The list is keyed to applicable section numbers of the practice):

X1.1 *Paragraph 5.4*—Maximum particle size if different from this section.

X1.2 *Sections 5 and 7 and Table 3*—Further restrictions on use of categories of embedment and backfill materials.

X1.3 *Paragraph 5.5*—Specific gradations of embedment materials for resistance to migration.

X1.4 *Paragraph 6.1.1*—State specific restrictions on leaving trenches open.

X1.5 *Paragraph 6.2*—Restrictions on mode of dewatering; design of underdrains.

X1.6 *Paragraph 6.3*—Requirements on minimum trench width.

X1.7 *Paragraph 6.4*—Restrictions or details for support of trench walls.

X1.8 *Paragraph 7.5*—Specific restrictions on methods of compaction.

X1.9 *Paragraph 7.5.1 and Table 3*—Minimum embedment density if different from these recommendations; specific density requirements for backfill (for example, for pavement subgrade).

X1.10 *Paragraph 7.6*—Minimum cover requirements if different from this paragraph.

X1.11 *Paragraph 7.7.1*—Detailed requirements for manhole connections.

X1.12 *Paragraph 7.7.2*—Detailed requirements for support of vertical risers, standpipes, and stacks to accommodate anticipated relative movements between pipe and such appurtenances. Detailing to accommodate thermal movements, particularly at risers.

X1.13 *Paragraph 8.11*—Requirements on methods of testing compaction and leakage.

X1.14 *Paragraph 8.12*—Requirements on deflection and deflection measurements, including method and time of testing.

X2. SOIL STIFFNESS

X2.1 In 2000, AASHTO adopted new values for soil stiffness for backfill materials used for thermoplastic pipe. The modifications include changing the soil design parameter from the modulus of soil reaction, E', to the constrained soil modulus, Ms. This change is based on the work of McGrath (1998).[9] Design values of the constrained modulus are presented in Table X2.1. The table shows that Ms increases with depth of fill which reflects the increased confining pressure. This is a well-known soil behavior. At moderate depths of fill the values of Ms are close to the E' values proposed by Howard (1977, 1996).[10,11] In design for deflection control, Ms may be

[9] McGrath, T. J., "Replacing E' with the Constrained Modulus in Flexible Pipe Design," *Proceedings of the Conference Pipelines in the Constructed Environment*, ASCE, 1998.

[10] Howard, A. K., "Modulus of Soil Reaction Values for Buried Flexible Pipe," *Journal of Geotechnical Engineering*, ASCE, Vol 103, No. GT1, New York, NY, 1977.

[11] Howard, A. K., *Pipeline Installation*, Relativity Publishing, Lakewood, CO, 1996.

TABLE X2.1 Constrained Soil Modulus, M_s Based on Soil Type and Compaction Condition, Stiffness Category and Vertical Stress[A,B,C]

Vertical Stress Level[D] (psi)	SC2-100 (psi)	SC2-95 (psi)	SC2-90 (psi)	SC2-85 (psi)
1	2,350	2,000	1,275	0,470
5	3,450	2,600	1,500	0,520
10	4,200	3,000	1,625	0,570
20	5,500	3,450	1,800	0,650
40	7,500	4,250	2,100	0,825
60	9,300	5,000	2,500	1,000
Vertical Stress Level[D] (psi)		SC3-95 (psi)	SC3-90 (psi)	SC3-85 (psi)
1		1,415	670	360
5		1,670	740	390
10		1,770	750	400
20		1,880	790	430
40		2,090	900	510
60		2,380	1,120	700
Vertical Stress Level[D] (psi)		SC4-95 (psi)	SC4-90 (psi)	SC4-85 (psi)
1		530	255	130
5		625	320	175
10		690	355	200
20		740	395	230
40		815	460	285
60		895	525	345

[A] The soil types are defined by a stiffness category (SC) to indicate the general soil classification (See AWWA Manual M45). Specific soil groups that fall into these categories, based on Classification D 2487 and AASHTO M 145, are listed in Table X2.2.

[B] The numerical suffix to the soil type indicates the compaction level of the soil as a percentage of maximum dry density determined in accordance with Test Method D 698 or AASHTO T-99.

[C] For design, dumped SC1 soils may use the modulus values of SC2–90 and compacted SC1 soils may use the values of SC2–100.

[D] Vertical stress level is the vertical effective soil stress at the springline elevation of the pipe. It is normally computed as the design soil density times the depth of fill to the springline. Buoyant soil density should be used below the groundwater level.

substituted directly for E' in the Iowa formula. Use of the constrained modulus in predicting deflection may be completed by making a direct substitution of Ms for E' in the Iowa formula. The Iowa formula is presented in many publications, in particular, AWWA Manual M45, Fiberglass Pipe Design.

X2.2 Example: Determine the constrained soil modulus at a depth of 10 ft for an SW soil with a unit weight of 120 pcf.

X2.2.1 Determine vertical applied stress:

p = 10 ft (120 pcf) = 1200 psf = 8.3 psi.

X2.2.2 Determine constrained soil modulus for SW soil, which is soil type SC2-100 from Table X2.1, note c:

for p = 5 psi, M_s= 3,450 psi;

for p = 10 psi, M_s= 4,200 psi.

X2.2.3 Interpolating for p = 8.3 psi, $M_s = 3450 + (4200 - 3450) \{(8.3 - 5)/(10 - 5)\} = 3945$ psi.

TABLE X2.2 Equivalent AWWA, ASTM, and AASHTO Soil Classifications

Soil Type[A]	ASTM D 2487	AASHTO M 145
SC1[B]	Crushed rock: 15 % sand, maximum 25 % passing the 3⁄8 in. sieve and maximum 5 % passing No. 200 seive	
SC2[C] (Gravelly sand, SW)	Clean, coarse grained soils: SW, SP, GW, GP or any soil beginning with one of these symbols, with 12 % or less passing No. 200 seive	A1, A3
SC3[C] (Sandy silt, ML)	Coarse grained soils with fines: GM, GC, SM, SC, or any soil beginning with one of these symbols, containing 12 % or more passing No. 200 seive; Sandy or gravelly fine-grained soils: CL, ML, (or CL-ML, CL/ML. ML/CL) with 30 % or more retained on a #200 sieve	A-2-4, A-2-5, A-2-6; or A-4 or A-6 soils with 30 % or more retained on a No. 200 sieve
SC4[C] (Silty clay, CL)	Fine-grained soils: CL, ML, (or CL-ML, CL/ML. ML/CL) with 30 % or less retained on a #200 sieve	A-2-7; or A-4 or A-6 soils with 30 % or less retained on a No. 200 sieve

[A] The soil classification listed in parentheses is the type that was tested to develop the constrained soil modulus values in Table X2.1. The correlations to other soil types are approximate.

[B] SC1 soils include crushed rock and gravels with limited sand content. For design, dumped SC1 soils may use the modulus values of SC2-90 and compacted SC1 soils may use the modulus values of SC2-100.

[C] Uniform fine sands (SP) with more than 50 % passing a No. 100 sieve (0.006 in., 0.15 mm) shall not be used as backfill for fiberglass pipe unless specifically allowed in the contract documents and special precautions are taken to control moisture content and monitor compaction levels.

NOTE—Soil type SC5, including MH, CH, and organic soils are not recommended for use as pipe backfill under any conditions.