Chapter 12

Horizontal Directional Drilling

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Introduction

The Horizontal Directional Drilling (HDD) Industry has experienced so much growth in the past few decades that HDD has become commonplace as a method of trenchless installation. According to Underground Infrastructure magazine (June 2025), HDPE is the most commonly installed pipe using HDD. This growth has been driven by the benefits offered to utility owners (such as the elimination of traffic disruption and minimal surface damage) and by the ingenuity of contractors in developing this technology. To date, HDD pipe engineering has focused on installation techniques, and rightfully so. In many cases, the pipe experiences its maximum lifetime loads during the pullback operation.

The purpose of this chapter is to inform the reader of some of the important considerations in selecting the proper PE pipe. Proper selection of pipe involves consideration not only of installation design factors such as pullback force limits and buckling/collapse resistance, but also of the long-term performance of the pipe once installed in the bore. The information herein is not all-inclusive; there may be effects not discussed that will have significant bearing on the proper engineering of an application and the pipe selection.

For specific projects, the reader is advised to consult with a qualified engineer to evaluate the project and prepare a specification including recommendations for design and installation and for pipe selection (e.g., MAB-11, “MAB Model Specifications for Installation of PE4710 Water Service, Distribution, and Transmission Pipes by Horizontal Directional Drilling”). The reader may find additional design and installation information in ASTM F1962, “Standard Guide for Use of Maxi-Horizontal Directional Drilling for Placement of Polyethylene Pipe or Conduit Under Obstacles, Including River Crossings,” the ASCE Manual of Practice 108, “Pipeline Design for Installation by Directional Drilling, and the NASTT “Horizontal

Directional Drilling Good Practices Guidelines." Regarding the actual construction process, properly trained crews can make the difference between a successful and an unsuccessful drilling program for a utility. Several institutions provide HDD operator-training programs, one of which is University of Texas at Arlington Center for Underground Infrastructure Research and Education (CUIRE).

Background

Some of the earliest uses of large diameter PE pipe in directional drilling were for river crossings. These are major engineering projects requiring thoughtful design, installation, and construction, while offering the owner the security of deep river bed cover with minimum environmental damage or exposure, and no disruption of river traffic. PE pipe is suited for these installations because of its scratch tolerance and the fused joining system which gives a zero-leak-rate joint with design tensile capacity equal to that of the pipe itself.

To date, directional drillers have installed PE pipe for gas, water, and sewer mains; communication conduits; electrical conduits; and a variety of chemical lines. These projects involved not only river crossings but also highway crossings and rights-of-was through developed areas so as not to disturb streets, driveways, and business entrances.

PE Pipe for Horizontal Directional Drilling

This chapter presents information on the pipe selection and design process. It is not intended to be a primer on directional drilling. The reader seeking such information can refer to the references of this chapter. Suggested documents are the "Mini-Horizontal Directional Drilling Manual" and the "Horizontal Directional Drilling Good Practices Guidelines" published by the North American Society for Trenchless Technology (NASTT). Also see "Trenchless Technology: Pipeline and Utility Design, Construction and Renewal," by McGraw Hill.

Horizontal Directional Drilling Process

General knowledge of the directional drilling method by the reader is expected, but some review may be of value in establishing common terminology. Briefly, the HDD process begins with boring a small, horizontal hole (pilot hole) under the crossing obstacle (e.g. a river or highway) with a continuous string of steel drill rod. When the bore head and rod emerge on the opposite side of the crossing, a special cutter, called a backreamer, is attached and pulled back through the pilot hole. The reamer expands the pilot hole so that the pipe can be pulled through with minimal resistance. The pipe is usually pulled through from the side of the crossing opposite the drill rig.

Pilot Hole

The drilling of the initial pilot hole is a critical step in the operation. The pilot hole establishes the path of the drill rod (drill-path) and subsequently the location of the PE pipe. Typically, the bore-head is tracked electronically to guide the hole along a pre-designed configuration. One of the key considerations in the design of the drill-path is avoiding unnecessary route bends and creating as large a radius of curvature as possible within the limits of the right-of-way, thus minimizing curvature. Route bends increase the pullback load due to the "capstan" effect and curvature induces bending stresses. (The capstan effect refers to the increase in frictional drag when pulling the pipe around a curve due to a component of the pulling force acting normal to the curvature, thereby increasing the bearing pressure.) Higher tensile stresses also reduce the pipe's collapse resistance. The drill-path normally has unavoidable curvature along its vertical profile. Curvature requirements are dependent on the site geometry (crossing length, required depth to provide safe cover, staging site location, etc.). However, the degree of curvature is limited by the bending radius of the drill rod and the pipe characteristics. (The ability to accomplish a sharp bend may depend upon the bearing capacity of the soil.) Typically, the permitted bending radius of the steel drill rod controls the curvature and thus significant bending stresses do not occur in the polyethylene (PE) pipe. The designer should maximize the radii of curvature by carefully choosing the entry and exit points. The driller should also attempt to minimize extraneous bends and associated curvature due to undulations resulting from path corrections.

Pilot Hole Reaming

Pilot hole reaming is as important to an HDD pipeline as backfill placement is to an open-cut pipeline. The reaming operation consists of using an appropriate tool to expand the pilot hole to a somewhat larger diameter than the product pipeline. The percentage oversize depends on many variables including soil types, soil stability, depth, drilling fluid (or slurry/mud), borehole hydrostatic pressure, etc. Typical over-sizing may be from 1.2 to 1.5 times the diameter of the product pipe. While the over-sizing is necessary to help minimize insertion forces, the inserted pipe will have to sustain external (soil, drilling fluid, groundwater) pressures without significant side support from the surrounding soil.

Prior to pullback of the pipe, a final reaming pass (swab pass) is often made using the same sized reamer as will be used when the pipe is pulled back. The swab pass cleans the borehole, removes remaining fine gravels or clay clumps and can compact the borehole walls.

Drilling Fluid

A drilling fluid is injected into the borehole during cutting and reaming to stabilize the hole and remove soil cuttings. Drilling fluid can be made from clay or polymers. The primary clay used for drilling fluid is sodium montmorillonite (bentonite). Properly ground and refined bentonite is added to fresh water to produce the drilling fluid. The drilling fluid aids in cutting, thereby reducing drilling torque, and mixes with cuttings to create a slurry or "mud" that gives stability and support to the bored hole. The fluid must have sufficient gel strength to keep cuttings suspended for transport, and to form a filter cake on the borehole wall that contains the water within the drilling fluid, while providing lubrication between the pipe and the borehole on pullback. Drilling fluids are designed to match the soil and cutter. They are monitored throughout the process to make sure the bore stays open, pumps are not overworked, and fluid circulation throughout the borehole is maintained. Loss of circulation could cause a locking up and possible overstressing of the pipe during pullback.

Drilling fluids are thixotropic and thus thicken when left undisturbed after pullback. However, depending upon the local soil conditions and tendency for the mud slurry to dehydrate, unless cementitious agents are added, the thickened mud may be no stiffer than very soft clay. In such cases, the mud may provide little to no soil side-support for the pipe.

Pullback

The pullback operation involves pulling the entire pipeline length (usually) in one segment back through the reamed pathway and the drilling mud. It is important that proper pipe handling, fusion procedures, cradling, bending minimization, and surface inspection, be followed and proper pull head connections shall be used. Axial tension force readings, constant insertion velocity, mud flow circulation/exit rates, and footage length installed should be recorded. The pullback speed typically varies from one to several feet per minute, depending on the size and local conditions. Excessive pullback speed should be avoided since this may result in "hydrolock", a condition where the available retraction force is not sufficient to overcome the pressure differential at the backreamer.

Horizontal Directional Drilling (HDD)

The industry distinguishes between mini-HDD and conventional HDD, which is sometimes referred to as maxi-HDD. Maxi-HDD technology is capable of installing pipes as large as 48 inch diameter or greater, in bore thousands of feet in length, at depths up to 200 feet. While the industry may differ somewhat in precise terminology, mini-HDD is typically limited to placing pipes up to approximately 12 inch diameter, at distances of hundreds of feet, at depths up to 15 feet. Mini-HDD rigs are used primarily for local distribution applications and utility construction in urban areas. Midi-HDD refers to systems with intermediate capability. Unless otherwise indicated, most of the information provided in this chapter is primarily applicable to the maxi-HDD operations, with mini-HDD discussed in a separate section below. Engineering judgement may be used to determine which information (maxi-HDD or mini-HDD) is appropriate for a particular midi-HDD installation.

General Guidelines

The designer will achieve the most efficient design for an application by consulting an experienced contractor and a qualified engineer. The following general considerations may help, particularly in regard to site location for PE pipes:

1. Select the crossing route to keep it to the shortest reasonable distance and/or in the most favorable soil conditions.
2. Locate routes and sites where the pipeline can be pre-constructed in one continuous length; or at least in long multiple segments to be fused together during the overall process.
3. Although compound curves are possible, the drill-path should be as straight as possible.
4. Avoid entry and exit elevation differences in excess of 50 feet; both points should be as close to the same elevation as possible.
5. Locate all buried structures and utilities within 10 feet of the drill-path for mini-HDD applications and within 25 feet of the drill-path for maxi-HDD applications. Crossing lines are typically exposed to confirm their exact location and depth.
6. Observe and avoid above-ground structures, such as power lines, which might limit the height available for construction equipment.
7. The HDD process takes very little working space relative to other methods. However, actual site space varies somewhat depending upon the crossing distance, pipe diameter, and soil type.
8. Long crossings with large diameter pipe will require larger equipment and more powerful drill rigs.
9. As pipe diameter increases, larger volumes of drilling fluids must be pumped, requiring more, or larger, pumps and mud-cleaning and storage equipment.
10. Space requirements for maxi-HDD rigs can range from a 100 feet wide by 150 feet long entry plot for a 1,000 ft crossing, and up to a 200 feet wide by 300 feet long area for a crossing of 3,000 or more feet.
11. On the opposite (pipe entry) side of the crossing, sufficient temporary space should be available to allow fusing and joining the PE carrier pipe in a continuous string, beginning about 75 feet beyond the exit point with a width of 35 to 50 feet, depending on the pipe diameter. Space requirements for coiled pipe are considerably less. Larger pipe sizes require larger and heavier construction

equipment which needs more maneuvering room (although use of flexible PE somewhat alleviates this requirement). The pipe entry side of the crossing should have an area approximately 50 feet wide by 100 feet long for most crossings, and up to 100 feet by 150 feet for large diameter crossings.

12. "As-built" drawings must be obtained based on the location of the actual completed drill-path. The as-built drawings are essential to know the exact pipeline location and to avoid future third party damage.

Safety

Safety is a primary consideration for every directionally drilled project. While this chapter does not cover safety, there are several manuals that discuss safety including the "Safety Manual: Directional Drilling Tracking Equipment"(3) provided by the Equipment Manufacturer's Institute (EMI), as well as the operator's manual provided by the manufacturer of the drill rig.

Geotechnical Investigation

Prior to any consideration of the pipe design or installation details, the responsible engineer or designer will usually conduct a comprehensive geotechnical study to identify soil formations and conditions at the potential bore sites. The purpose of the investigation is not only to determine if directional drilling is feasible, but to establish the most efficient way to accomplish the task, including determination of the optimum drilling fluid for the soil conditions. With this geotechnical information, the optimum crossing route can be determined, drilling tools and procedures selected, and the pipe type and size determined. The extent of the geotechnical investigation often depends on the pipe diameter, bore length and the nature of the crossing. Refer to ASTM F1962, "Guide for Use of Maxi-Horizontal Directional Drilling for Placement of Polyethylene Pipe or Conduit Under Obstacles, Including River Crossings"(4), ASCE MOP 108, "Pipeline Design for Installation by Horizontal Directional Drilling"(5) and the NASTT "Horizontal Directional Drilling Good Practices Guidelines"(2) for additional information.

During the study, the geotechnical consultant should identify a number of relevant items including the following:

- The presence of rock, rock inclusions, gravelly soils, loose deposits, discontinuities and hardpan
- Soil strength and stability characteristics
- Groundwater

Supplemental geotechnical data may be obtained from existing records; e.g. recent bridge constructions, or other pipeline/cable crossings in the area.

For long crossings, borings are typically taken at 700-ft intervals. For short crossings (1,000 ft or less), as few as three borings may suffice. The borings should be near the drill-path to give accurate soil data, but sufficiently far from the borehole to avoid pressurized mud from following natural ground fissures and rupturing to the ground surface through the soil-test borehole. A rule-of -thumb is to take borings at least 30 ft to either side of bore path. Borings somewhat deeper than the initially planned path may prove useful in the event that unanticipated problems arise at the planned elevation. However, the number, depth and location of boreholes is best determined by the geotechnical engineer.

Geotechnical Data for River Crossings

River crossings require additional information including a study to identify river bed, river bed depth, stability (lateral as well as scour), and river width. Typically, pipes are installed to a depth of at least 20 ft below the expected future river bottom, considering scour. Soil borings for geotechnical investigation are generally conducted to 40 ft below river bottom.

Summary

The best conducted projects are handled by a team approach with the owner's design engineer, bidding contractors and geotechnical engineer participating prior to the preparation of contract documents. The geotechnical investigation is usually the first step in the boring project. Once the geotechnical investigation is completed, a determination can be made to confirm whether HDD can be used. At that time, design of both the PE pipe and the installation can begin. The preceding paragraphs represent general guidance and considerations for planning and designing an HDD PE pipeline project. In addition, some projects may require special permits for sensitive areas (e.g., wetlands, federal highways, levees, …) which can significantly affect the planned path and/or procedures. These overall topics can be very detailed in nature. Individual HDD contractors and consultant engineering firms should be contacted and utilized in the planning and design stage. A rational in-depth analysis of all pertinent considerations should prevail. Care should be given in evaluating and selecting an HDD contractor based upon successful projects, qualifications, experience and diligence. A team effort, strategic partnership and risk-sharing is recommended.

Product Design: PE Pipe DR Selection

After completion of the geotechnical investigation and determination that HDD is feasible, the designer selects the appropriate PE pipe. The proper pipe must satisfy all hydraulic requirements of the line including flow capacity, working pressure rating, and surge or vacuum capacity. These considerations have to be met regardless of the method of installation. Design of the pipe for hydraulic considerations can be found in Chapter 6. For HDD applications, in addition to the hydraulic requirements, the pipe must be able to withstand (1) pullback loads, which primarily include tensile pull forces and external hydrostatic pressure, and (2) external service loads, including post-installation soil, groundwater, and surcharge loads occurring over the life of the pipeline.

PE pipe is classified by its “dimension ratio” (DR) which is defined as the pipe’s outer diameter divided by the minimum wall thickness. A more detailed explanation of the DR concept is provided in Chapter 5.

Often the load the pipe experiences during installation, such as the combined pulling force and external pressure due to the slurry (drilling fluid plus cuttings), will be the largest load experienced by the pipe during its life. The remainder of this chapter will discuss the DR selection based on pullback and external service loads.

While this chapter provides guidelines to assist the designer, the designer assumes all responsibility for determining the appropriateness and applicability of the equations and parameters given in this chapter for any specific application. Directional drilling is an evolving technology, and industry-wide design protocols are still developing. Proper design requires considerable professional judgment beyond the scope of this chapter. The designer is advised to consult ASTM F 1962 when preparing an HDD design. This methodology has been applied, for example, for designing the municipal water pipe crossings described by Petroff(6).

In general, the designer begins the DR selection process by determining the DR requirement to withstand the internal pressure. The designer will then determine if this DR is sufficient to withstand earth, live, and groundwater service loads (Chapter 6). An internal vacuum, however, must be combined with the external pressure. If the DR selection can withstand these loads, then the installation (pullback) forces are considered. Ultimately, the designer chooses a DR that will satisfy all three requirements: pressure, service loads, and pullback load.

Although there can be some pipe wall stresses generated by the combination of internal pressurization and wall bending, internal pressure and external service load stresses are generally treated as independent. However, the internal pressurization significantly reduces stresses due to external loads by re-rounding. The effects of these stresses are reflected in the limits provided in Table 2, for pressure applications. Wall bending may be caused by localized bearing forces, but the local stresses associated with the latter do not require a separate analysis. This is permissible primarily since PE is a ductile material and failure is usually driven by the average stress rather than local maximums.

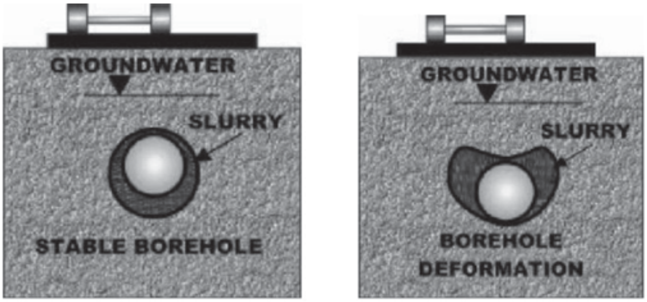


Figure 1 Borehole Deformation

Design Considerations for Net External Loads

This section and the following sections discuss external buried loads that may be imposed on pipes installed by directionally drilling (HDD). One important factor in determining what load reaches the pipe is the condition of the borehole; i.e. whether it remains round and open or deforms or collapses (see Figure 1). This will depend on a great extent on the type of soil, the boring techniques, and the presence of the slurry (drilling fluid and cutting mixture). If the borehole does not deform (remains round) after drilling, earth loads are arched around the borehole and little or no soil pressure is transmitted to the pipe. The pressure acting on the pipe is then the hydrostatic pressure due to the slurry or any groundwater present. The slurry itself may act to keep the borehole open. If the borehole collapses, or deforms substantially, earth pressure will be applied to the pipe. The resulting pressure could exceed the slurry pressure unless considerable tunnel arching occurs above the borehole. Where no tunnel arching occurs, the applied external pressure is equal to the combined earth, groundwater, and live-load pressure. For river crossings, in unconsolidated river bed soils, little arching is anticipated and the applied soil pressure likely equals its geostatic stress (i.e., the "prism load" of the soil). In consolidated soils, arching above the borehole may occur, and the applied pressure will likely be less than the geostatic stress, even after total collapse of the borehole crown onto the pipe. If the soil deposit is a stiff clay, cemented, or partially lithified, the borehole may stay open with little or no deformation. In this case, the applied pressure is likely to be just the slurry head or groundwater head. If it may be assumed that the mud eventually dehydrates and solidifies, the pipe will be supported by the surrounding "soil", providing constraint and increasing its resistance to collapse.

In addition to the overt external pressures such as slurry head and groundwater, internal vacuum in the pipe results in an effective increase in external pressure due to the removal of atmospheric pressure from inside the pipe. On the other hand, a positive internal pressure in the pipe may mitigate some or all of the external pressure. The following equations can be used to establish the net external pressure, or differential pressure between the inside and outside of the pipe.

Depending on the borehole condition, the net external pressure is defined by either Eq. 1 (deformed/collapsed borehole) or Eq. 2 (open borehole):

(1)

$$P_N = P_E + P_{GW} + P_{SUR} - P_I$$

(2)

$$P_N = P_{MUD} - P_I$$

WHERE

P_N = net external pressure, psi

P_E = earth pressure, psi

P_{GW} = groundwater pressure (including the height of surface water), psi

P_{SUR} = surcharge and live loads, psi

P_I = internal pressure, psi (negative in the event of vacuum)

P_{MUD} = hydrostatic pressure of slurry (or groundwater), psi

The earth, groundwater, surcharge and mud (slurry) pressures in Eqs. 1 and 2 are discussed below.

When calculating the net external pressure, the designer should give consideration to all possible applied loads and their duration, including surges. Most pipelines go through operational cycles that include (1) unpressurized or being drained, (2) operating at working pressure, (3) flooding, (4) shutdowns, and (5) vacuum and peak pressure events. As each of these cases could result in a different net external pressure, the designer should consider all phases of the line's life to establish the various design cases.

In addition to determining the load, careful consideration should be given to the duration of each load. PE pipe is viscoelastic; i.e., its effective properties depend on duration of loading. For example, an HDD conduit resists constant groundwater and soil pressure with its long-term apparent stiffness modulus. Alternatively, an HDD force-main may be subjected to a sudden vacuum resulting from water hammer. When a vacuum occurs, the net external pressure equals the sum of the external pressure plus the vacuum. Since this surge is instantaneous, it is resisted by the pipe's short-term apparent stiffness modulus, which may be several times greater than the long-term apparent modulus.

For pressure lines, consideration should be given to the time the line sits unpressurized following construction. This may be several months. Most directionally drilled lines that contain liquid will have a static head (internal pressure), which will remain in the line once filled. This head may be subtracted from the external pressure due to earth/groundwater loading. The designer should also consider that the external load also may vary with time, such as due to flooding.

Mud, Earth and Groundwater Pressure

Earth loads can reach the pipe when the borehole deforms and contacts the pipe. The amount of soil load transmitted to the pipe, and resulting pipe deflection, will depend on the extent of borehole deformation and the relative stiffness between the pipe and the soil. Groundwater loading will occur whether the hole deforms or not, although may be of less magnitude than that due to the slurry.

The designer may wish to consult a geotechnical engineer for assistance in determining earth and groundwater loads, as the loads reaching the pipe depend on the nature of the soil and the stability of the borehole.

Stable Borehole – Mud or Groundwater Pressure Only

A borehole is considered stable if it remains round, with minimal deformation, subsequent to completion of the drilling operation, for the design period of interest. For instance, drilling in competent rock (rock that can be drilled without fracturing and collapsing) will typically result in a stable borehole; see Figure 1. Stable boreholes may occur in some soils where the slurry exerts sufficient pressure to maintain a round and open hole. Since the associated deformations around the hole are small, soil pressures transmitted to the pipe are negligible. The external load applied to the pipe consists only of the hydrostatic pressure due to the slurry and/or groundwater, if present. Equation 3 provides the hydrostatic pressure due to drilling mud (slurry) and Eq. 4 provides the hydrostatic pressure due to groundwater. Standing surface water should be added to the height of groundwater.

(3)

$$P_{MUD} = \frac{\gamma_{MUD} \, H_{MUD}}{144}$$

(4)

$$P_{GW} = \frac{\gamma_W \, H_W}{144}$$

WHERE

P_{MUD} = **hydrostatic pressure of slurry, psi**

γ_{MUD} = **weight of slurry (drilling mud and cuttings), lb/ft^3**

H_{MUD} = **elevation difference between lowest point in borehole and (lower elevation of) entry or exit pit, ft**

P_{GW} = **groundwater pressure (including the height of surface water), psi**

γ_W = **weight of water, lb/ft^3**

H_W = **height of water level above pipe, ft**

Borehole Deforms/Collapse with Arching Mobilized

When the crown of the hole deforms sufficiently to place soil above the hole in the plastic state, arching is mobilized. In this state, hole deformation is limited. If no soil touches the pipe, there is no earth load on the pipe. However, when deformation is sufficient to transmit load to the pipe, it becomes necessary to determine the degree of earth load applied to the pipe.

Based on the successful performance of directionally drilled PE pipes, it is reasonable to accept that some amount of arching occurs in most applications. It may also be assumed that the approaches developed for determining earth pressure on auger bored pipes and on jacked pipes is somewhat relevant to pipes installed by directional drilling. Thus, O'Rourke et al.(7) published an equation for determining the earth pressure on auger bored pipes assuming a borehole approximately 10% larger than the pipe. In this model, arching occurs above the pipe similar to that in a tunnel where zones of loosened soil fall onto the pipe. The volume of the cavity is eventually filled with soil that is slightly less dense than the insitu soil, but still capable of transmitting soil load. This method of load calculation gives a minimal loading. The method published herein, however, is more conservative. It is based on trench type arching as opposed to tunnel arching and is used by Stein(8) to calculate loads on jacked pipe. In Stein's model, the maximum earth load (effective stress) is found using the modified form of Terzaghi's equation given by Eq. 5 and Eq. 6. (For additional information on post-installation design of directionally drilled pipelines, see Petroff(9)). Stein and O'Rourke's methods should only be considered where the depth of cover is sufficient to develop arching (typically exceeding five pipe diameters), dynamic loads such as traffic loads are insignificant, the soil has sufficient internal friction to transmit arching (i.e., not saturated), and conditions are confirmed by a geotechnical engineer.

Using the equations given in Stein, the external pressure is given below:

(5)

$$P_E = \frac{k \gamma_S H_C}{144}$$

(6)

$$k = \frac{1 - \exp\left[-2 \frac{KH_C}{B} \tan\left(\frac{\delta}{2}\right)\right]}{2 \frac{KH_C}{B} \tan\left(\frac{\delta}{2}\right)}$$

WHERE

K = **earth pressure coefficient, given by:**

$$K = \tan^2\left(45 - \frac{\varphi}{2}\right)$$

and

P_E = **earth pressure, psi**

γ_S = **weight of soil, lb/ft^3**

H_C = **depth of soil cover, ft**

k = **arching factor**

B = **"silo" width, ft**

δ = **angle of wall friction, degrees (for HDD, $\delta = \varphi$)**

φ = **angle of internal friction, degrees**

The "silo" width, B, should be estimated based on the application. It varies between the pipe diameter and the borehole diameter. A conservative approach is to set the silo width to equal the borehole diameter. For water level below the ground surface, the buoyant weight of the saturated soil (prism load), plus the groundwater pressure, must be added to the above pressure on the pipe.

Borehole Collapse with Prism Load (No Arching)

In the event that arching in the soil above the pipe breaks down, considerable earth loading may occur on the pipe. In the event that arching does not occur, the load is the weight of the soil prism above the pipe. The prism load is most likely to develop in shallow applications subjected to live or dynamic loads, and boreholes in unconsolidated sediments such as in some river crossings, as described below. The "prism" load is given by Eq. 7, without the arching factor, k; or:

(7)

$$P_E = \frac{\gamma_S H_C}{144}$$

WHERE

P_E = **earth pressure, psi**

γ_S = **weight of soil, lb/ft^3**

H_C = **depth of soil cover, ft**

For water level below the ground surface, the buoyant weight of the saturated soil (prism load), plus the groundwater pressure, must be added to the above pressure on the pipe.

Combination of Earth and Groundwater Pressure (No Arching)

In the absence of arching, and where groundwater is present, the prism loads will apply, in addition to groundwater pressure. For instance, in a river crossing one can have reasonable confidence that the directionally drilled pipe is subjected to the earth pressure from the sediments above combined with the water pressure, as indicated below:

Case 1 Water level at or below ground surface

(8)

$$P_E + P_{GW} = \frac{\gamma_B H_W + \gamma_S (H_C - H_W) + \gamma_W H_W}{144}$$

Case 2 Water level at or above ground surface (i.e. pipe in river bottom)

(9)

$$P_E + P_{GW} = \frac{\gamma_B H_C + \gamma_W H_W}{144}$$

WHERE

P_E = earth pressure, psi

γ_S = weight of soil, lb/ft^3

γ_W = weight of water, lb/ft^3

γ_{SAT} = weight of saturated soil, lb/ft^3

γ_B = buoyant weight of soil = $\gamma_{SAT} - \gamma_W$, lb/ft^3

H_C = height of soil cover, ft

P_{GW} = groundwater pressure (including the height of surface water), psi

H_W = height of water level above pipe, ft

> **Note:** The portion of loading due to the asymmetric soil pressure on the pipe, P_E, causing ring deflection, is obtained by deleting the terms containing γ_W on the right sides of Eqs. 8 and 9.

Live Loads

Wheel loads from trucks or other vehicles are significant for pipe at shallow depths whether they are installed by open cut trenching or directional drilling. The wheel load applied to the pipe depends on the vehicle weight, the tire pressure and size, vehicle speed, surface smoothness, pavement and distance from the pipe to the point of loading. In order to develop proper soil structure interaction, pipe subject to vehicular loading should be installed at least 18 inches or one pipe diameter (whichever is greater) under the road surface. HDD installed pipes are typically installed at a deeper depth in order to prevent inadvertent returns of drilling fluid from occurring during the boring operation.

The soil pressure due to live load, such as an H20 wheel or E-80 railroad, can be found in Tables 3-3, 3-4 and 3-5 in Chapter 6, or can be calculated using one of the methods in Chapter 6. To determine the total pressure applied to the pipe, the soil pressure due to live load, P_L, is added to the earth pressure, P_E. See Example 1 in Appendix A.

Performance Limits

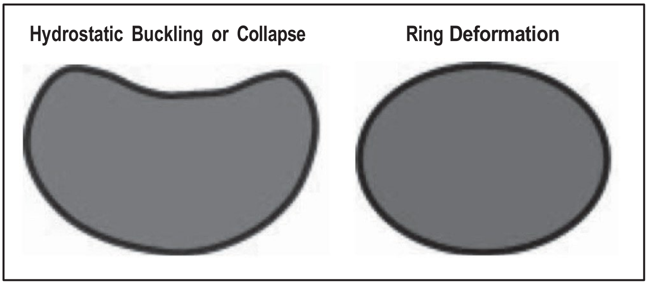


Figure 2 Performance Limits of HDD Pipe Subjected to Service Loads

Performance Limits of HDD Installed Pipe

External pressure from earth load, surface or ground water, vacuum and live load applied to the HDD pipe produces (1) a compressive ring thrust in the pipe wall and possibly (2) ring bending deflection. The performance limit of unsupported PE pipe subjected to compressive thrust is ring buckling (collapse). The performance limit of a PE pipe subjected to ring bending (a result of non-uniform external load, such as earth loading) is ring deflection. See Figure 2.

Viscoelastic Behavior

Both performance limits are proportional to the apparent modulus of elasticity of the PE material. For viscoelastic materials such as PE, the modulus of elasticity is a time (and temperature) dependent property; i.e., its value changes with time under load. An applied load increment will result in a decrease in apparent stiffness over time. Unloading will result in rebounding at an apparently larger stiffness. In general, there will be a higher resistance to short term loading than to long-term loading. Careful consideration must be given to the duration and frequency of each load, so that the performance limit associated with that load can be calculated using PE material properties representative of that time period. Similar effects occur with the pipe's tensile strength. For instance, during pullback, the pipe's tensile yield strength decreases with pulling time, so the safe (allowable) pull stress is a function of time under load. The safe pull stress and apparent elastic modulus also depend on temperature, with both properties increasing at lower temperatures.

TABLE 1
Safe Pull Tensile Stress @ 80°F

Duration (Hours)	Typical Safe Pull Stress (psi) @ 80°F		Time under tension design factor per ASTM F1804, Note 4
	PE2708 (commonly used for gas pipe)	PE4710	
0.5	1040	1400	1.0
1	1040	1400	1.0
12	988	1330	0.95
24	946	1274	0.91
ASTM D3350 Cell Classification	PE234373	PE445574	
Minimum tensile yield strength (psi) per ASTM D3350	2600	3500	
HDD tensile yield design factor per ASTM F1804	0.4	0.4	

The safe pull stress values shown in the table may be considered to be associated with complete strain recovery after completion of the pullback and a suitable recovery period.

Typical safe pull tensile stress values for MDPE and HDPE are provided in Table 1. However, it is recommended that the manufacturer be consulted for specific applications. The values in Table 1 are given as a function of the load duration. For pipe temperatures (which are not necessarily the outside air temperatures) other than 80ºF, the values in Table 1 should be multiplied by the temperature compensating factor in Table B.1.2 of the Appendix to Chapter 3. The corresponding safe pull force at 12 hours effective load duration is provided for a variety of pipe sizes and DR values in Tables 3 and 4 (PE4710 material). The safe pull force for 1 hour effective duration is provided in Tables 5 and 6 (PE4710 material).

Ring Deflection (Ovalization)

Non-uniform pressure acting on the pipe's circumference, such as earth load, causes bending deflection of the pipe ring. Normally, the deflected shape is an oval (Figure 2). Initial ovalization may also exist in non-rerounded coiled pipe and to a lesser degree in straight lengths that have been stacked, but the primary sources of long-term bending deflection of directionally drilled pipes is that due earth load. Slight ovalization may also occur during pullback if the pipe is pulled around a curved path in the borehole. Ovalization reduces the pipe's hydrostatic collapse resistance and creates bending stresses in the pipe wall. It is normal and expected for belowground PE pipes to undergo ovalization, whether installed in trenches or by direction drilling. Proper design and installation will limit ovalization (i.e., "ring deflection") to prescribed values so that it has no adverse effect on the pipe during its lifetime.

Ring Deflection Due to Earth Load

As discussed previously, insitu soil characteristics and borehole stability determine to a great extent the earth load applied to directionally drilled pipes. Methods for calculating estimated earth loads, when they occur, are given above.

Since earth load is non-uniform around a pipe's circumference, the pipe will undergo ring deflection; i.e. a decrease in vertical diameter and an increase in horizontal diameter. The soil surrounding the pipe may help resist the deflection of the pipe under the earth load. Related formulas used for entrenched pipe, such as Spangler's Iowa Formula(10), are likely not applicable as the HDD installation is different from installing pipe in a trench where the embedment can be controlled.

In an HDD installation, the annular space surrounding the pipe contains a mixture of drilling fluid and cuttings. Unless the mixture tends to dehydrate and solidify, its consistency determines the possible degree of resistance which depends on several factors including soil density, grain size and the presence of groundwater. It is noted that researchers have excavated pipe installed by HDD and observed some tendency of the annular space soil to return to the condition of the undisturbed native soil. (See Knight(11) and Ariaratnam(12)) These installations, however, were located above groundwater, where excess water in the mud-cuttings slurry can drain. Therefore, while there may be consolidation and strengthening of the soil in the annular space in some conditions, particularly above the groundwater level, it may be weeks or even months before significant resistance to pipe deflection develops. Until further research establishes the soil's contribution to resisting deflection, it is conservative to ignore any soil resistance and to use Equation 10 which is derived from ring deflection equations published by Watkins and Anderson (13). (Equation 10 gives the same deflection as the Iowa Formula with an E′ of zero. Spangler's Iowa formula is discussed in Chapter 6.) Note that the apparent modulus of elasticity is a function of the duration of the anticipated load.

The design deflection limits for directionally drilled pipe are provided in Table 2.

(10)

$$\frac{\Delta y}{OD} = 0.15\,(DR - 1)^3\ P_E/E$$

WHERE

Δy = **vertical ring deflection, in.**
OD = **pipe outside diameter, in.**
P_E = **earth pressure, psi**
E **= apparent elastic modulus, psi (see Appendix B, Chapter 3, Material Properties)**
DR = **pipe Dimension Ratio**
Note: To obtain ring deflection in percent, multiply $\Delta y/OD$ by 100.

TABLE 2
Design Deflection Limits of Buried Polyethylene Pipe, Long Term*

DR	21	17	13.5	11	9	7.3
Deflection Limit (% Δy/OD) Non-Pressure Applications	7.5	7.5	7.5	7.5	7.5	7.5
Deflection Limit (% Δy/OD) Pressure Applications	7.5	6.0	6.0	5.0	4.0	3.0

* Design deflection limits per ASTM F1962, "Standard Guide for Use of Maxi-Horizontal Directional Drilling for Placement of Polyethylene Pipe or Conduit Under Obstacles, Including River Crossings".

Unconstrained Buckling

(See Chapter 6 for a detailed discussion of buckling for an unconstrained pipe.)

External pressure applied to the pipe either from earth and live load, groundwater, or the slurry creates a ring compressive hoop stress in the pipe's wall. If the external pressure is increased to a point where the compressive hoop stress reaches a critical value, the pipe will collapse with a sudden and large inward deformation of the pipe wall: i.e., "buckling". Constraining the pipe by embedding it in soil or cementitious grout will increase the pipe's buckling strength and allow it to withstand higher external pressure than if unconstrained. However, as indicated above, it is not likely that pipes installed below the groundwater level will acquire significant support from the surrounding slurry mixture, and it may take considerable time for significant support to develop for pipe installed above groundwater level. Therefore, as discussed above, it is conservative to assume the soil provides no constraint. However, when the pressure within the pipe exceeds the external pressure due to earth and live load, groundwater and/or slurry, the stress in the pipe wall reverses from compressive to tensile stress and collapse will not occur.

The following equation, known as Levy's equation, may be used to determine the allowable net external pressure (or negative internal pressure) for unconstrained pipe, to avoid buckling.

(11)

$$P_{UC} = \frac{2\,E}{(1-\mu^2)}\left(\frac{1}{DR-1}\right)^3 \frac{f_o}{N}$$

WHERE

P_{uc} = **allowable net unconstrained pressure, psi**

E = **apparent elastic modulus, psi; see Appendix B, Chapter 3, Material Properties)**

μ = **Poisson's Ratio = 0.45 for all PE pipe materials**

DR = **pipe Dimension Ratio: OD/t, where t = minimum wall thickness**

f_0 = **ovality compensation factor; see Figure 3,** f_0 = **(1 - fraction ovality)**^{9}

N = **safety factor (generally 2.0 or greater)**

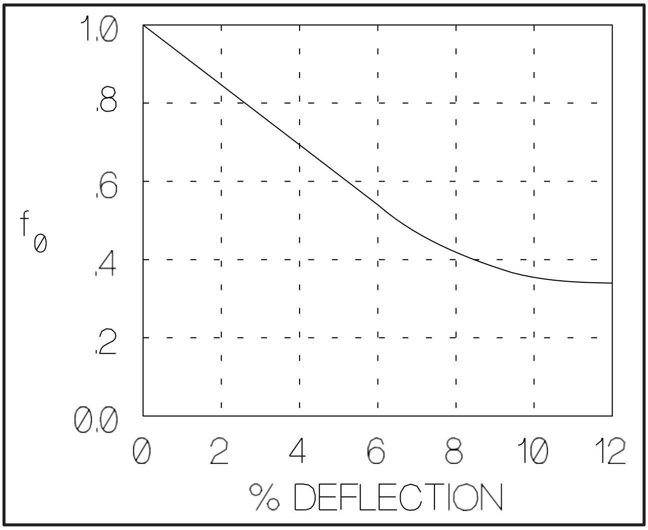


Figure 3 Ovality Compensation Factor, f_0,= (1 – fraction ovality)^9

For determining the pipe's resistance to buckling during pullback, an additional reduction for tensile stresses is required, which is discussed below when considering the installation forces.

Wall Compressive Stress

The compressive stress itself in the wall of a directionally drilled PE pipe rarely controls design and it is generally not considered. However, in some special cases, such as directional drilling at very deep depths, it may control design.

The earth pressure applied to a buried pipe creates a compressive stress in the pipe wall. When the pipe is pressurized, the stress is reduced due to the internal pressure creating opposing tensile stresses. The net stress may be positive or negative, depending on the depth of cover. Buried pressure lines may be subject to net compressive stress when shut down, or especially when experiencing vacuum. These are usually short-term conditions and are not typically considered significant for design, since the short-term design strength of polyolefins is considerably higher than the long-term value.

Pipes with a large depth of cover and operating at low pressures may have a net compressive stress in the pipe wall. The following equation may be used to determine the net compressive stress:

(12)

$$S_C = \frac{P_E \times OD - P_I \times ID}{2\,t}$$

WHERE

S_C = **compressive wall stress, psi**
P_E = **earth pressure, psi**
P_I = **internal pressure, psi**
OD = **pipe outside diameter, in.**
ID = **pipe inside diameter, in.**
t = **minimum wall thickness: OD/DR in.**

The compressive wall stress should be kept less than the allowable compressive stress of the material. See Table C.1 of Appendix C of Chapter 3 for a safe allowable stress.

EXAMPLE CALCULATIONS Example calculations for selecting or confirming the DR for an HDD pipe, based on service (post-installation) loads, are given in Appendix A.

Installation Design Considerations

In addition to determining the DR required for long-term service (post-installation), the installation loads must be considered. A lower DR (stronger pipe) may be required to withstand the installation forces.

During pullback, the pipe is subjected to axial tensile forces required to overcome the frictional drag between the pipe and the borehole and ground surface or support system, outside the borehole, amplified by the “capstan” effect around path bends, plus hydrokinetic drag. In addition, the pipe is subjected to the external pressure of the external drilling fluid/slurry head. The pipe’s collapse resistance to the external pressure given in Eq. 3 is reduced by the axial tensile force. There are also some bending stresses at route bends but are relatively low due to the typically large path curvatures, and torsional forces are usually negligible when backreamer swivels are properly employed.

Considerable judgment and experience is required to predict the pullback force because of the complex interaction between pipe and soil. Sources for information include experienced drillers and engineers, programs such as PPI-BoreAid™(14) (https://www.ppiboreaid.com/) and publications such as ASTM F1962, ASCE MOP 108, and the NASTT Good Practices Guidelines. The methods in the latter two documents are also based on ASTM F1962 for the installation of PE pipe. In general, pullback force calculations are approximations that are based on the ground conditions and path profile. For an overview of the general procedure, see Svetlik(15).

The pullback formulas given herein are based on ASTM F1962 and assume essentially an "ideal" bore. The ideal borehole behaves as a stable tunnel with smooth alignment (no inadvertent bends), complete cuttings removal, and good slurry circulation. The ideal borehole may be approached with proper drilling techniques that achieve a clean bore, fully reamed to its final size prior to, or during, pullback.

Because of the large number of variables involved and the sensitivity of pullback forces to installation techniques, the methodology presented in ASTM F1962 and in this document are provided primarily to familiarize the designer with the interaction that occurs during pullback. The corresponding results should be considered as qualitative values and used only for preliminary estimates. For a typical large-scale "maxi-HDD" operation it is essential to consult with an experienced driller and/or a knowledgeable engineer.

Pullback Force – Maxi-HDD

Typical large HDD (maxi-HDD) rigs can exert a pull force of 100,000 lbs to 500,000 lbs, and possibly much greater. Much of the power is applied to the cutting face of the reamer device/tool during the preliminary or subsequent reaming operations. During the pullback of the pipe, the total pull force applied by the rig must be sufficient to pull the reamer, the drill rods, and the pipe. Therefore, the force indicated at the drill rig generally exceeds that experienced by the pipe itself. The tension at the pipe must not exceed the safe pull strength of the pipe.

The pulling force on the pipe must overcome the frictional drag, internal and external to the borehole, aggravated by the capstan effect at bends, plus that due to hydrokinetic (fluidic) drag. The axial tensile stress in the pipe is a maximum at the leading (pulling) end. The differential stress intensity along the length of the pipeline causes a varying degree of recoverable elastic strain and viscoelastic stretch per foot of length along the pipe.

The DR must be selected so that the maximum tensile stress in the pipe wall due to the pullback force, does not exceed the permitted tensile stress for the pipe material. Increasing the pipe wall thickness (lower DR value) will allow for a greater total pull force. Although the thicker wall increases the weight per foot of the pipe, the pullback force within the bore itself is not significantly reduced by the increased weight because the dominant frictional forces are due to the large upward buoyancy effects on the pipe, which are only partially offset by the (downward) weight of the pipe itself.

Frictional Drag

Frictional drag results from the normal (perpendicular) pressure of the force between the pipe and the borehole surface, and between the pipe and the ground surface or support system external to the borehole, in the entry area. Equation 13 gives the frictional resistance or required pulling force for pipe pulled in straight level bores, or along a level aboveground surface. (See Kirby et al. (16)).

(13)

$$F_P = v\, w_N\, L$$

WHERE

F_P = **pulling force, lbs**

v = **coefficient of friction between pipe and local surface**

  v_a = coefficient of friction applicable at the surface before the pipe enters bore hole. Per ASTM F1962 the suggested design value is 0.5; where pipe is placed on rollers v_a is considered equal to 0.1

  v_b = coefficient of friction applicable within the lubricated bore hole or after the (wet) pipe exits. Per ASTM F1962 the suggested design value is 0.3

w_N = **net downward or upward normal force on pipe at contact surface, lb/ft**

L = **segment length, ft**

When a slurry is present in the borehole, w_N equals the upward buoyant force on the pipe minus the weight of the pipe and its contents, if any. Filling the pipe with fluid (e.g., water) significantly reduces the buoyancy and thus the surface friction and required pulling force. PE pipe has a density near that of water. If the pipe is installed empty, using a closed pull head, the pipe will float, resting against the crown of the borehole. The high local surface pressure combined with the coefficient of friction against the borehole surface results in significant frictional drag. Therefore, most major pullbacks are performed with the pipe filled with water as it descends into the bore. Water is added through a hose or small pipe inserted into the pipe.

> **Note:** The buoyant force pushing the empty pipe up against the crown of the borehole will cause the PE pipe to rub against the borehole surface. During pullback, the slurry

lubricates the contact area, and the pipe is subject to kinetic (sliding) friction. When the drilling operation or pipe movement stops, the subsequent start-up ("static") friction is greater than the sliding friction. The resulting pulling load can be higher than otherwise estimated. If considered necessary, this situation may be addressed by conservatively using a thicker (lower DR) pipe and/or implementing the above anti-buoyancy techniques, and stopping the pull only when removing drill rods.

Capstan Effect

For curves along the borehole, Huey et al.(17) considers an additional frictional force that occurs when installing steel pipe, due to the normal reaction forces at the borehole surface required to conform the relatively stiff steel pipe to the curve. For bores with large radii of curvature required by the steel drill rods, these additional forces are insignificant for the flexible PE pipe. However, there is another source of increased friction at route bends or curvature that is independent of pipe stiffness or weight (or buoyancy), or whether the bends are sharp or gradual. This is the principle of the capstan winch, and is based on the increased surface pressure at bends due to the tension vectors tending to pull a rope into the convex surface of the rotating drum (capstan). Similarly, as a pipe is pulled around a curve or bend, such as those shown in the HDD geometry of Figure 4, there is an amplification and compounding of the forces along the route. For example, the pulling force, F_C, resulting from amplification of the force of Eq. 13 is given in Eq. 14.

(14)

$$\begin{aligned} F_C &= e^{\nu\,\theta}\,\nu\,w_N\,L \\ &= \exp(\nu\,\theta)\,\nu\,w_N\,L \end{aligned}$$

WHERE

e = **Natural logarithm base (e=2.71828)**
F_C = **pulling force amplified by capstan effect, lbs**
ν = **coefficient of friction between pipe and local surface**
w_N = **net upward or downward normal force on pipe at contact surface, lb/ft**
L = **segment length, ft**
θ = **angle of bend, radians**

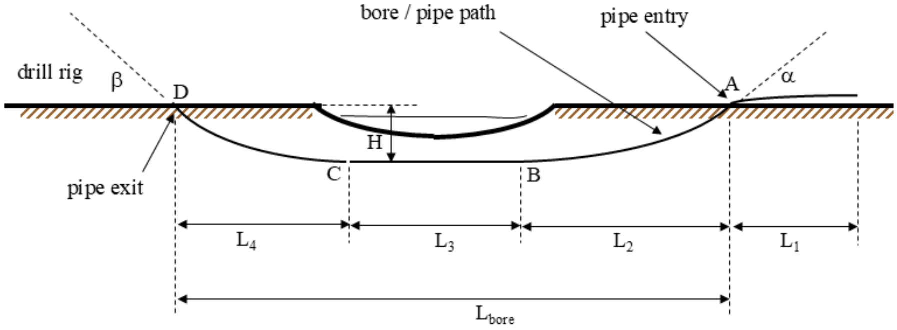


Figure 4 HDD Geometry

Sequential application of Eqs. 13 and 14 along the borehole path of Figure 4, results in the following set of equations:

(15)

$$F_A = \exp(v_g\,\alpha)\left[v_g\,w_p\,(L_1 + L_2 + L_3 + L_4)\right]$$

$$F_B = \exp(v_b\,\alpha)\left[F_1 + v_b\,w_b\,L_2 + w_b\,H - v_g\,w_p\,L_2\exp(v_g\,\alpha)\right]$$

$$F_C = F_2 + v_b\,w_b\,L_3 - \exp(v_b\,\alpha)\left[v_g\,w_p\,L_3\exp(v_g\,\alpha)\right]$$

$$F_D = \exp(v_b\,\beta)\left\{F_3 + v_b\,w_b\,L_4 - w_b\,H - \exp(v_b\,\alpha)\left[v_g\,w_p\,L_4\exp(v_g\,\alpha)\right]\right\}$$

WHERE

F_i = **pullback force on pipe at point i, lbs**
L_i = **horizontal distance of pull from point to point, ft**
H = **depth of bore, ft**
v_g = **coefficient of friction above ground**
v_b = **coefficient of friction in borehole**
w_p = **weight of (empty) pipe, lb/ft^3**
w_b = **net upward normal force on pipe within borehole, lb/ft^3**
α = **pipe entry angle, radians**
β = **pipe exit angle, radians**

These equations are consistent with those in ASTM F1962 and assume that the entry and exit points of the borehole are at the same elevation.

Hydrokinetic (Fluidic) Drag

During the pullback, pipe movement is also resisted by the fluidic drag force imposed by the slurry. Hydrokinetic pressure is required to pump drilling mud from the reamer into the borehole and annulus, and along the pipe length while conveying reamed soil debris to the mud recovery pit. This hydrokinetic pressure, and the associated drag imposed on the pipe, is difficult to estimate and depends on the drilling fluid, slurry flow rate pipe pullback rate, and the borehole and pipe sizes. The corresponding hydrokinetic pressure is estimated to be in the 4 to 8 psi range(15), and is only present during the operation of the reamer. ASTM F1962 attempts to account for the general magnitude of this drag component, considered to be an additional, or incremental force, as follows:

(16)

$$\Delta_F = P_{HK}\frac{\pi}{8}\left({D_h}^2 - OD^2\right)$$

WHERE

Δ_F = **hydrokinetic drag, lbs**
P_{HK} = **hydrokinetic pressure, psi**
D_h = **borehole diameter, in.**
OD = **pipe outside diameter, in.**

The fluidic drag estimate of Eq. 16 is separately added to each F_i term of Eq. 15, but is not intended to be cumulative. ASTM F1962 conservatively suggests a hydrokinetic pressure of 10 psi for the present purposes.

ASCE MOP 108 suggests a different method for calculating the hydrokinetic drag force. This alternative was originally developed for application to the installation of steel pipe, and proposes multiplying the external surface area of the pipe within the borehole by a fluid drag coefficient of 0.025 lb/in^2, as in Puckett$^{(18)}$. This estimate is of much greater magnitude than that of Eq. 16. In general, the ASTM F1962 methodology, represented by Eqs. 15 and Eq. 16, has been accepted within the industry for application to PE pipe, and is illustrated in the examples at the end of this chapter. During pullback it is advisable, as possible, to monitor the pulling force, and/or to use a breakaway link (with swivel), or other equivalent method, to prevent over-stressing the pipe.

Tensile Stress During Pullback

The maximum outer fiber tensile stress should not exceed the safe pull stress. This maximum outer fiber tensile stress is obtained by taking the sum of the maximum tensile force in the pipe due to the frictional drag force of Eqs. 15, plus the fluidic drag contribution of Eq. 16, and the tensile bending stress due to pipe curvature.

The peak tensile stress occurring in the pipe wall during pullback is therefore given by Eq. 17, which includes the major contribution of the maximum pulling force, F_{max}, plus the relatively low bending stress.

(17)

$$\sigma_T = \sigma_{avg} + \sigma_{bend}$$

WHERE

$$\sigma_{avg} = \frac{F_{max}}{\pi\, t\,(OD - t)}$$

$$\sigma_{bend} = \frac{E \times OD}{2\,R}$$

and

σ_T = **peak tensile stress, psi**

σ_{avg} = **average (cross-sectional) maximum tensile stress, psi**

σ_{bend} = **axial bending stress, psi**

F_{max} = **maximum pulling force, including hydrokinetic drag (Eq. 16), lbs**

OD = **pipe outside diameter, in.**

t = **minimum wall thickness: OD/DR, in.**

E = **apparent elastic modulus, psi (see Appendix B, Chapter 3, Material Properties)**

R = **minimum radius of curvature in bore path. at point of maximum pulling force, in.**

The peak axial tensile stress due to the pulling force should not exceed the pipe's safe pull stress. As discussed in a previous section, time under load is an important consideration in selecting the appropriate tensile strength to use in calculating the safe pull stress. During pullback, the pulling force is not continually applied to the pipe, as the driller must stop the pulling operation to extract each drill rod from the drill string. Pullback is therefore an incremental process rather than a continuous process, such that the pipe is not subjected to a constant tensile force and thus may relax some between pulls. A relatively short duration value for the applied stress might then be considered safe for design; however, a 12-hour value is reasonable or conservative and will normally minimize or eliminate possible "stretching" of the pipeline. Tables 3 and 4 provide safe pull forces for PE pipes based on a 12-hour value (1330 psi safe pull stress). The safe pull force, also referred to as "allowable tensile load", in these tables is based on the minimum pipe wall thickness and may be determined using Eq. 18. (Some applications may prefer to use the somewhat greater pull forces corresponding to the average wall thickness, rather than the minimum value. In such cases, the manufacturer should be consulted for the average wall values.) Allowable safe pullback values for gas pipe are provided in ASTM F-1807, "Practice for Determining Allowable Tensile Load for Polyethylene (PE) Gas Pipe during Pull-In Installation".

(18)

$$F_S = \sigma_S \, \pi \, OD^2 \left(\frac{1}{DR} - \frac{1}{DR^2} \right)$$

WHERE

F_S = **safe pull force, lbs**

σ_S = **safe pull stress, psi**

OD = **pipe outside diameter, in.**

DR = **Dimension Ratio**

After pullback, the pipe may take several hours (typically equal to the duration of the pull) to recover from the temporary axial strain. The leading end should be pulled out approximately 3% longer than the total length of the pull. The elastic strain will recover immediately and the viscoelastic stretch will "remember" its original length and recover overnight. This step is essential to avoid the leading end being retracted into the borehole resulting from recovery of the full temporary stretching, and possible thermal contraction to an equilibrium temperature. In some cases, to be conservative, the driller may pull out approximately 4% extra length (40 feet per 1000 feet) to ensure the leading end remains extended beyond the pipe exit from the borehole.

TABLE 3

PE 4710 12-hour Safe Pull Strength* (1330 psi stress) vs. IPS Size

		Safe Pull Force, lbs				
Size	**Nom. OD**	**7**	**9**	**11**	**13.5**	**17**
1.25	1.660	1,410	1,137	952	790	637
1.5	1.900	1,847	1,490	1,247	1,035	835
2	2.375	2,886	2,328	1,948	1,616	1,305
3	3.500	6,267	5,055	4,230	3,511	2,834
4	4.500	10,360	8,357	6,993	5,803	4,684
6	6.625	22,460	18,110	15,160	12,580	10,150
8	8.625	38,060	30,700	25,690	21,320	17,210
10	10.750	59,130	47,690	39,910	33,120	26,730
12	12.750	83,170	67,090	56,140	46,590	37,600
14	14.000	100,300	80,880	67,680	56,170	45,340
16	16.000	131,000	105,600	88,400	73,360	59,220
18	18.000	165,800	133,700	111,900	92,850	74,950
20	20.000	204,700	165,100	138,100	114,600	92,530
22	22.000	247,600	199,700	167,100	138,700	112,000
24	24.000	294,700	237,700	198,900	165,100	133,200
26	26.000	345,900	279,000	233,400	193,700	156,400
28	28.000	401,100	323,500	270,700	224,700	181,400
30	30.000	460,500	371,400	310,800	257,900	208,200
32	32.000	523,900	422,600	353,600	293,500	236,900
34	34.000	591,400	477,100	399,200	331,300	267,400
36	36.000	663,100	534,800	447,500	371,400	299,800
42	42.000		728,000	609,100	505,500	408,100
48	48.000		950,800	795,600	660,300	533,000
54	54.000			1,007,000	835,700	674,500
63	63.000			1,371,000	1,137,000	918,100

*Table values are based on the minimum wall thickness of pipe

TABLE 4
PE 4710 12-hour Safe Pull Strength* (1330 psi stress) vs. DIPS Size

		Safe Pull Force, lbs				
Size	**Nom. OD**	**7**	**9**	**11**	**13.5**	**17**
3	3.96	8,023	6,471	5,415	4,494	3,628
4	4.80	11,790	9,508	7,956	6,603	5,330
6	6.90	24,360	19,650	16,440	13,640	11,010
8	9.05	41,900	33,800	28,280	23,470	18,950
10	11.10	63,040	50,850	42,550	35,310	28,500
12	13.20	89,150	71,900	60,170	49,930	40,310
14	15.30	119,800	96,600	80,830	67,090	54,150
16	17.40	154,900	124,900	104,500	86,760	70,040
18	19.50	194,500	156,900	131,300	109,000	87,960
20	21.60	238,700	192,500	161,100	133,700	107,900
24	25.80	340,600	274,700	229,900	190,800	154,000
30	32.00	523,900	422,600	353,600	293,500	236,900
36	38.30	750,500	605,300	506,500	420,400	339,300
42	44.50		817,200	683,800	567,500	458,100
48	50.80		1,065,000	891,100	739,600	597,000
54	57.56		1,367,000	1,144,000	949,500	766,400
60	61.61		1,566,000	1,311,000	1,088,000	878,100

*Table values are based on the minimum wall thickness of pipe

External Pressure During Installation

During pullback it is reasonable to assume that the borehole remains stable and open and that the borehole is filled with (mud), resulting in an external hydrostatic pressure due to the height of the slurry above the pipe, P_{MUD}. This external pressure can be offset by allowing the slurry to enter the pipe, or filling the pipe with water, during the pullback operation. However, this may not always be desirable or possible, such as when installing electrical conduit containing pre-installed wires or cables.

In addition to the hydrostatic pressure of the slurry in the borehole, there are also dynamic sources of external pressure:

1. If the pulling end of the pipe is capped, a plunger action occurs during pulling which creates a mild surge pressure. The pressure is difficult to calculate. The pipe will resist such an instantaneous pressure with its relatively high short-term modulus. If care is taken to pull the pipe smoothly, at a constant speed, this effect is minimized and may be ignored. Alternatively, if the pulling end of the pipe is left open, this surge is also eliminated.

2. Hydrokinetic pressure, as described above, is required to pump drilling mud into the borehole and convey reamed soil debris to the mud recovery pit. The hydrokinetic pressure is conservatively estimated at 10 psi in ASTM F1962.

In consideration of the dynamic or hydrokinetic pressure, Eq. 2 is modified to account for this additional load during the installation phase:

(19)

$$P_N = P_{MUD} + P_{HK} - P_I$$

WHERE

P_N = **net external pressure, psi**
P_{HK} = **hydrokinetic pressure, psi**
P_I = **internal pressure, psi (negative in the event of vacuum)**
P_{MUD} = **hydrostatic pressure of slurry, psi**

Resistance to External Collapse Pressure During Pullback Installation

The external buckling pressure equation, Eq. 11, with the appropriate apparent modulus value, and a reduction in collapse strength due to axial tension, can be used to calculate the pipe's resistance to the external pressure, P_N, given by Eq. 19 during pullback.

The pulling force on the pipe results in a hoop strain as determined by Poisson's ratio. The hoop strain reduces the resistance to buckling, which may be. as accounted for by a reduction factor, f_r. shown in Eq. 20 and calculated in Eq. 21.

(20)

$$P_{tens} = \frac{2\,E}{(1-\mu^2)} \left(\frac{1}{DR-1}\right)^3 \frac{f_o\, f_r}{N}$$

WHERE

P_{tens} = **allowable unconstrained pressure, under tension, psi**
E = **apparent elastic modulus, psi (see Appendix B, Chapter 3, Material Properties)**
μ = **Poisson's Ratio = 0.45 for all PE pipe materials**
DR = **pipe Dimension Ratio: OD/t, where t = minimum wall thickness**
f_o = **ovality compensation factor; see Figure 3, or (1-fraction ovality)[9]**
f_r = **tension factor (Eq. 21)**
N = **safety factor (generally, 2.0 or greater)**

(21)

$$f_r = \sqrt{[5.57 - (r + 1.09)^2]} - 1.09$$

and

$$r = \frac{\sigma_{avg}}{2\,\sigma_S}$$

for

σ_{avg} = **calculated tensile stress during pullback, psi (Eq. 17)**
σ_S = **safe pull stress, psi**

Since the pullback time is typically several hours, a modulus value consistent with the pullback time (e.g., 12 hours) can be selected from Appendix B Chapter 3.

Note: The procedure used for determining the ability of the PE pipe to withstand the external pressures, as reflected in Eqs. 19 – 21, as applied in some of the Examples, is conservative, and tends to overestimate the simultaneous loads actually experienced by the pipe, and/or underestimate the appropriate apparent elastic modulus. This is particularly the case when considering the portion of the pipe when at the maximum depth, when most vulnerable to collapse, due to the hydrostatic pressure of the mud/slurry and hydrokinetic pressure. At this point, a calculation of the tension factor, f_r, based on the peak pullback force, which usually occurs at the leading end of the pipe at the end of the pull (Figure 4, point D), will result in a lower collapse strength than necessary. Similarly, assuming the duration of the hydrokinetic pressure to be the same as the full pull time underestimates the value of the apparent elastic modulus, thereby underestimating the collapse strength of the pipe. PPI-BoreAid, discussed below, therefore makes reasonable engineering judgements in selecting more appropriate values in its interpretation and application of ASTM F1962.

EXAMPLE CALCULATIONS Example calculations for selecting or confirming the DR for an HDD pipe, based on installation or post-installation (service) loads, are given in Appendix A and B.

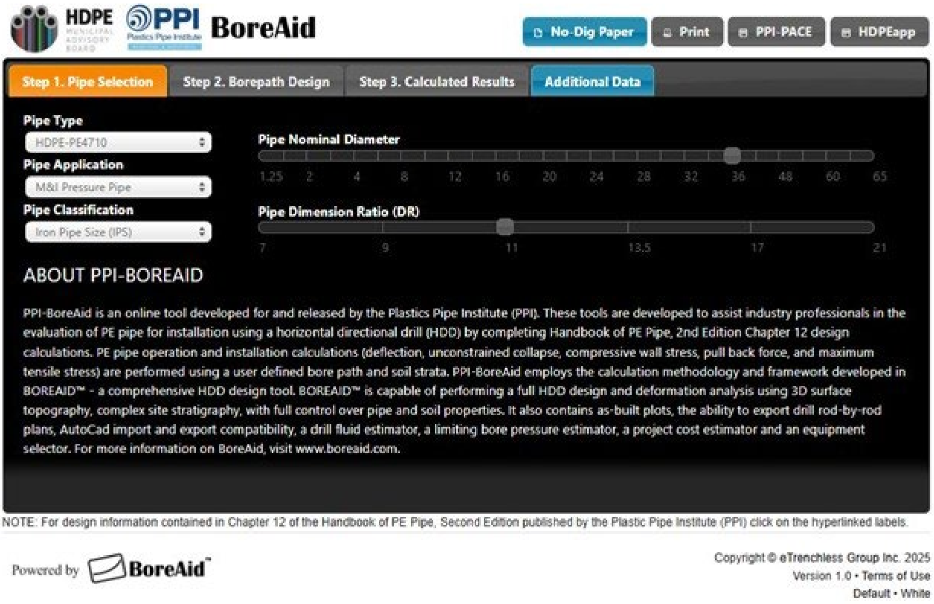
PPI-BoreAid™

The software tool PPI-BoreAid™ is available for facilitating the initial planning of the HDD operation for installing polyethylene pipe. The tool performs the calculations specified in ASTM F1962 for estimating the pull forces, as indicated in Figure 4. PPI-Boreaid also evaluates the potential for buckling or collapse during the installation phase as well as post-installation (operation/service), considering the long term stability of the installed pipe.

The calculations within PPI-BoreAid utilize the quantitative values of the significant parameters (e.g., coefficients of friction, drilling fluid density) as suggested in ASTM F1962. PPI-BoreAid selects appropriate values of the physical properties (e.g., elastic modulus, safe pull stress) which are dependent upon the effective (cumulative) durations of various loads on the pipe. The engineer, however, loses the ability to select other values of interest, or possibly deemed more appropriate, as in conducting initial studies. This is illustrated in the Example in Appendix B.

Figures 5 – 8 show screen shots of the PPI-BoreAid pages for describing the specific application and the results. The details of the PE pipe, and its application, are selected as shown in Figure 5:

- Pipe type/material (e.g., PE4710)
- Application (pressure or non-pressure/gravity sewer)
- Size classification system (IPS, DIPS)
- Nominal diameter
- Dimension Ratio (outer diameter divided by minimum thickness)



Pipe Type: HDPE-PE4710

Pipe Application: M&I Pressure Pipe

Pipe Classification: Iron Pipe Size (IPS)

Pipe Nominal Diameter: 36

Pipe Dimension Ratio (DR): 11

ABOUT PPI-BOREAID

PPI-BoreAid is an online tool developed for and released by the Plastics Pipe Institute (PPI). These tools are developed to assist industry professionals in the evaluation of PE pipe for installation using a horizontal directional drill (HDD) by completing Handbook of PE Pipe, 2nd Edition Chapter 12 design calculations. PE pipe operation and installation calculations (deflection, unconstrained collapse, compressive wall stress, pull back force, and maximum tensile stress) are performed using a user defined bore path and soil strata. PPI-BoreAid employs the calculation methodology and framework developed in BOREAID™ - a comprehensive HDD design tool. BOREAID™ is capable of performing a full HDD design and deformation analysis using 3D surface topography, complex site stratigraphy, with full control over pipe and soil properties. It also contains as-built plots, the ability to export drill rod-by-rod plans, AutoCad import and export compatibility, a drill fluid estimator, a limiting bore pressure estimator, a project cost estimator and an equipment selector. For more information on BoreAid, visit www.boreaid.com.

NOTE: For design information contained in Chapter 12 of the Handbook of PE Pipe, Second Edition published by the Plastic Pipe Institute (PPI) click on the hyperlinked labels.

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Figure 5 PPI-BoreAid: Select Pipe and Application Details

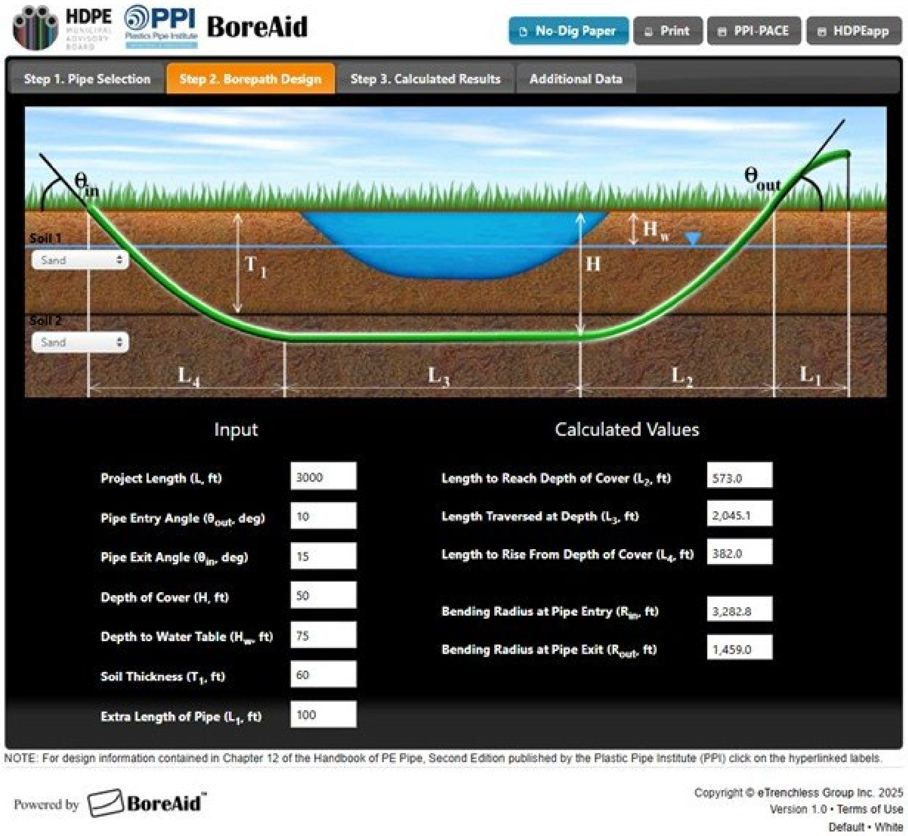
Figure 6 shows the details of the selected bore path:

- Bore length
- Entry and exit angles (i.e., α, β)
- Depth of cover (H)
- Soil and groundwater depths
- Soil types
- Extra pipe length (L_1)

Corresponding values for the segments L_2, L_3, and L_4, as well as the (average) bend radii at pipe entry and exit, are then calculated, using the formulas in ASTM F1962.

PPI-BoreAid evaluates the ability of the specified pipe (wall thickness, DR), to withstand both the installation and operational (service) loads. As indicated in the software tool, these are preliminary calculations only. Qualified professionals should be contracted to consider all aspects of the design for horizontal directional drilling.

Three different installation techniques are considered, each of which may be appropriate for a maxi-HDD operation: (1) pipe pulled back empty (vacant), tail end supported on ground surface, (2) pipe pulled back empty (vacant), tail end supported on (low friction) rollers, and (3) pipe pulled back filled with water (ballast), tail end supported on (low friction) rollers. The first technique corresponds to the most severe installation condition, with the third method the most favorable. The use of rollers and/or (especially) ballast reduce the required pullback force, which also has a positive effect on the collapse strength. These three categories are shown in Figures 7 and 8. Within each category, PPI-BoreAid evaluates the pipe strength (DR value) as specified, and also the next stronger size (lower DR) and the next weaker size (higher DR).



Soil 1: Sand

Soil 2: Sand

Input

Project Length (L, ft)	3000
Pipe Entry Angle (θ_{out}, deg)	10
Pipe Exit Angle (θ_{in}, deg)	15
Depth of Cover (H, ft)	50
Depth to Water Table (H_w, ft)	75
Soil Thickness (T_1, ft)	60
Extra Length of Pipe (L_1, ft)	100

Calculated Values

Length to Reach Depth of Cover (L_2, ft)	573.0
Length Traversed at Depth (L_3, ft)	2,045.1
Length to Rise From Depth of Cover (L_4, ft)	382.0
Bending Radius at Pipe Entry (R_{in}, ft)	3,282.8
Bending Radius at Pipe Exit (R_{out}, ft)	1,459.0

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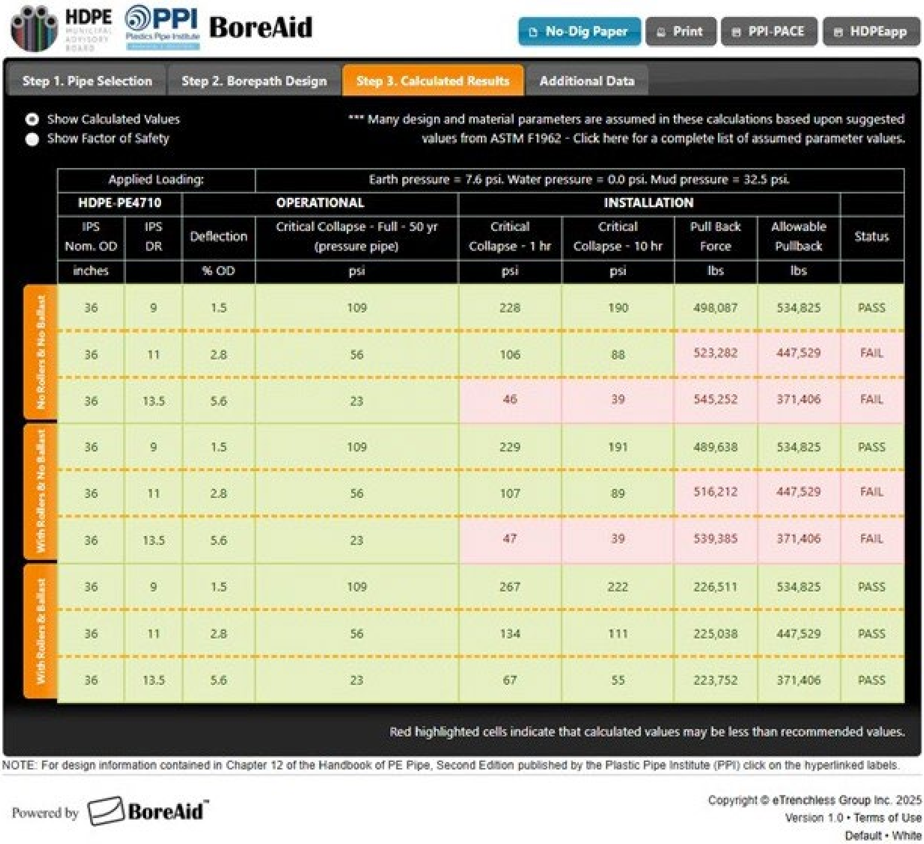
Figure 6 PPI-BoreAid: Specified Bore Path

Figure 7 shows the results based on the strength of the pipe in comparison to the calculated pull back loads and pressures applied to the pipe. During the installation phase, PPI-BoreAid considers the pressures applied during pullback, due to the hydrostatic pressure of the relatively dense slurry and the hydrokinetic pressure created by the backreamer. PPI-BoreAid also considers the long-term stability of the pipe, during the operational phase, when subject to earth and groundwater pressures, over a 50-year service life. The appropriate value of the allowable pullback load is based on the assumed tensile load duration (i.e., 12 hours). Similarly, the critical collapse pressures are based on the apparent elastic moduli corresponding to the indicated load durations. These collapse pressures are determined using formulas consistent with those of ASTM F1962, and as described in this chapter.

> **Note:** PPI-BoreAid uses the physical properties specified in Table X1.1 of ASTM F1962. This includes the 12-hour safe pull stress and the 10-hour apparent modulus of elasticity. Although these time periods are slightly different, there are only small or negligible quantitative differences in each of these properties for these two durations.

Since the collapse pressure depends on the effective load duration, PPI-BoreAid calculates both a 1-hour and 10-hour collapse strength, considering that the hydrokinetic pressure is only applied while the reamer is active (e.g., 1-hour cumulative duration). The 10-hour collapse strength is determined using 75% of the maximum tension for calculating the tension factor, f_r, (Eq. 21), since this is a more reasonable value to use when the pipe is at maximum depth, where it is most vulnerable to collapse. The long-term operational collapse strength is reduced by the ovality in the pipe – either an initially assumed 3% ovality or that due to the asymmetric soil loading, based on the characteristics of the specified soils and groundwater conditions relative to the depth of the pipe, whichever is greater. The ability to conveniently determine the effects of various soil and groundwater conditions is particularly useful.



◉ Show Calculated Values
○ Show Factor of Safety

*** Many design and material parameters are assumed in these calculations based upon suggested values from ASTM F1962 - Click here for a complete list of assumed parameter values.

	Applied Loading:		Earth pressure = 7.6 psi. Water pressure = 0.0 psi. Mud pressure = 32.5 psi.						
	HDPE-PE4710		OPERATIONAL		INSTALLATION				
	IPS Nom. OD	IPS DR	Deflection	Critical Collapse - Full - 50 yr (pressure pipe)	Critical Collapse - 1 hr	Critical Collapse - 10 hr	Pull Back Force	Allowable Pullback	Status
	inches		% OD	psi	psi	psi	lbs	lbs	
No Rollers & No Ballast	36	9	1.5	109	228	190	498,087	534,825	PASS
	36	11	2.8	56	106	88	523,282	447,529	FAIL
	36	13.5	5.6	23	46	39	545,252	371,406	FAIL
With Rollers & No Ballast	36	9	1.5	109	229	191	489,638	534,825	PASS
	36	11	2.8	56	107	89	516,212	447,529	FAIL
	36	13.5	5.6	23	47	39	539,385	371,406	FAIL
With Rollers & Ballast	36	9	1.5	109	267	222	226,511	534,825	PASS
	36	11	2.8	56	134	111	225,038	447,529	PASS
	36	13.5	5.6	23	67	55	223,752	371,406	PASS

Red highlighted cells indicate that calculated values may be less than recommended values.

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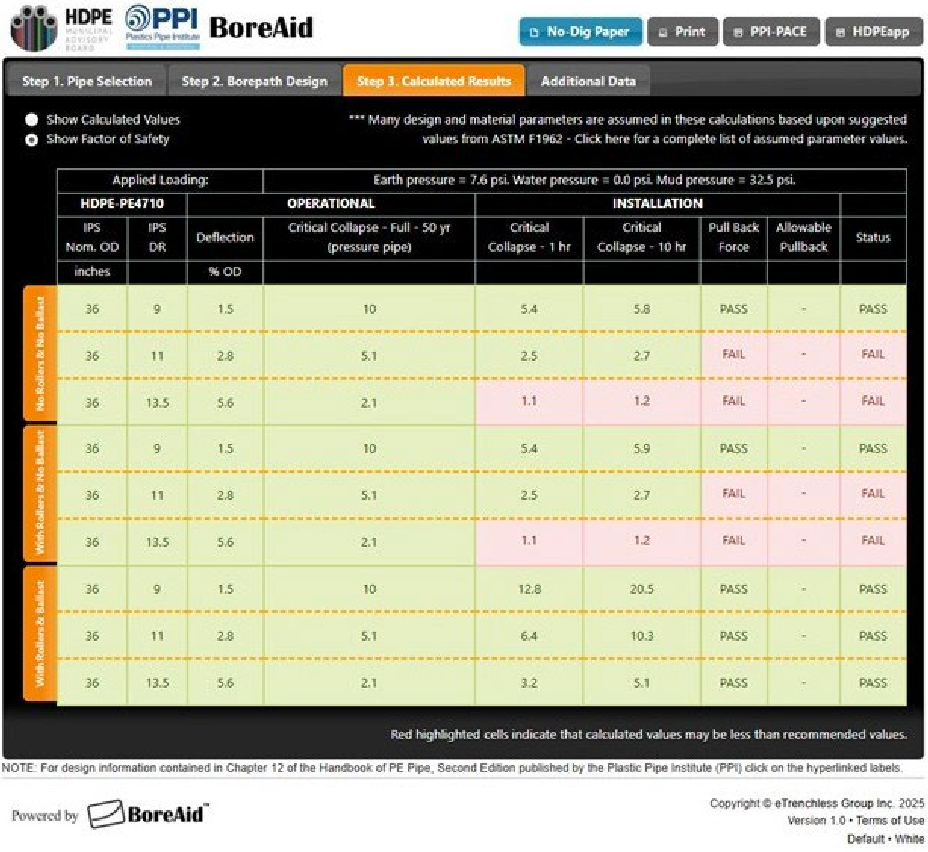
Figure 7 PPI-BoreAid: Results (Pipe Strengths vs. Effects of Loads)

Figure 8 is an optional alternative to Figure 7, and shows the corresponding factors of safety to withstand collapse, or just a PASS/FAIL indication for pullback force. In both Figures 7 and 8, a failure condition is highlighted in red.

While PPI-BoreAid is an extremely useful tool for performing initial or preliminary investigations to help ensure a successful large-scale, maxi-HDD installation, it is generally not necessary for a mini-HDD installation. Indeed, as discussed below, the load calculations based on PPI-BoreAid (or ASTM F1962) may significantly underestimate the pullback force actually experienced during a typically less well-controlled mini-HDD operation.



○ Show Calculated Values
● Show Factor of Safety

*** Many design and material parameters are assumed in these calculations based upon suggested values from ASTM F1962 - Click here for a complete list of assumed parameter values.

Applied Loading: Earth pressure = 7.6 psi. Water pressure = 0.0 psi. Mud pressure = 32.5 psi.

	HDPE-PE4710			OPERATIONAL	INSTALLATION				
	IPS Nom. OD inches	IPS DR	Deflection % OD	Critical Collapse - Full - 50 yr (pressure pipe)	Critical Collapse - 1 hr	Critical Collapse - 10 hr	Pull Back Force	Allowable Pullback	Status
No Rollers & No Ballast	36	9	1.5	10	5.4	5.8	PASS	-	PASS
	36	11	2.8	5.1	2.5	2.7	FAIL	-	FAIL
	36	13.5	5.6	2.1	1.1	1.2	FAIL	-	FAIL
With Rollers & No Ballast	36	9	1.5	10	5.4	5.9	PASS	-	PASS
	36	11	2.8	5.1	2.5	2.7	FAIL	-	FAIL
	36	13.5	5.6	2.1	1.1	1.2	FAIL	-	FAIL
With Rollers & Ballast	36	9	1.5	10	12.8	20.5	PASS	-	PASS
	36	11	2.8	5.1	6.4	10.3	PASS	-	PASS
	36	13.5	5.6	2.1	3.2	5.1	PASS	-	PASS

Red highlighted cells indicate that calculated values may be less than recommended values.

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Figure 8 PPI-BoreAid: Results (Factors of Safety)

Bending Stress

HDD river crossings incorporate route bends and curvature consistent with the large required bending radii of the steel drill rods used to install the PE pipe. These dimensions are well within the bending capability of PE pipe. For example, DR 11 PE pipe can be cold bent to 25 times its nominal OD; i.e., a 24 in DR 11 PE pipe may be bent to a minimum radius of 50 feet, or a 100-foot diameter circle, which is greatly exceeded by the path created by the steel drill rods. In order to minimize the effect of ovaling, some manufacturers limit the radius of curvature to a minimum of 40 to 50 times the pipe diameter, but such limits are still well within that imposed on the pipe within the borehole. Nonetheless, the relatively low tensile stress due to the pipe bending is included in the design calculations of Eq. 17.

Thermal Stresses and Strains

During pipe installation, the pipeline is not axially restrained by the oversize borehole. HDD pipelines generally become fully restrained in the axial direction as progressive sedimentation and soil consolidation occur within the borehole. This condition develops in the vast majority of cases. The rate at which restraint occurs depends on the soil and drilling techniques and can vary from a few hours to months.

The degree to which the pipeline will be restrained after completed installation is in large part a function of the sub-surface soil conditions and behavior, and the soil pressure at the depth of installation. Although the longitudinal displacement due to thermal contraction is minimal, it is in addition to the axial strain from stretching. Thus, as previously explained, the PE pipe should be cut to length only after it has reached thermal equilibrium with the surroundings and recovered from the temporary stretching (usually overnight).

Torsion Stress

A typical value for torsional shear strength is 50% of the tensile strength. The torsional shear stress imposed on the pipe may be calculated by dividing any possible transmitted torque by the wall area.

During the pullback and reaming procedure, a swivel is used to isolate the rotating cutting head assembly from the pipeline itself to minimize any torque being transmitted to the pipe. Swivels, however, are not 100% efficient and some fraction will be transmitted to the pipeline. Therefore, the size or rating of the swivel should not be excessively greater than the strength of the pipe being installed. For PE pipes of DR 17 or thicker, this torsion is generally not significant and usually does not require a detailed engineering analysis.

Mini-HDD

The preceding information and methodologies are generally applicable to large-scale HDD — i.e., maxi-HDD — operations. This technology is required for crossing large obstacles, such as wide rivers, and requires careful planning and coordination. In addition to the general guidelines described above, a comprehensive geotechnical study will usually be performed to help ensure a successful installation. In contrast, small-scale, mini-HDD operations, typically used for local distribution applications, must be cost-effective, and do not require similar extensive planning and design efforts. Nonetheless, safety remains a priority, and adherence to the local "call before dig" rules is essential. The contractor must provide advance notification to the "One-Call" center, or equivalent, to allow the various utilities to locate and mark their facilities in the vicinity of the intended construction. Physical contact with adjacent utilities, including gas and electric lines, must be avoided.

Maxi-HDD operations are supported by experienced, qualified engineers. In contrast, mini-HDD contractors may install several lines a day, usually without the benefit of an engineer, and simpler procedures are therefore appropriate. MAB-7, "MAB Guidelines for Use of Mini-Horizontal Directional Drilling for Placement of HDPE (PE4710) Pipe in Municipal Applications", provides such a methodology, including a convenient means of confirming the appropriate pipe strength for such applications.

Buckling

The commonly used wall thicknesses, with the possible exception of DR 17 pipe, are generally sufficiently strong for depths of 15 ft, the typical limit for mini-HDD installations, or greater. DR 17 pipe should generally be limited to less than 10 ft depth, although 15 ft may also be acceptable in some cases.(19) The design examples in Appendix A, demonstrate the ability to withstand the pressures at these relatively shallow depths.

Pullback Force

As an alternative to the relatively complicated methodology for estimating the pullback force for the maxi-HDD operations, as provided above, it is recognized that the dominant effect determining the required pulling force is the frictional drag associated with the buoyant weight of the HDPE pipe within the borehole. Anti-buoyancy techniques, such as filling the pipe with water to reduce the buoyancy, are not typically employed in mini-HDD operations. Therefore, the peak pull force will always occur at or near point D (Figure 4), the end of the pull, and is directly related to the length of the bore. By selecting reasonable and/or conservative values for the parameters in Eqs. 15, some as suggested within ASTM F1962, and making various mathematical approximations, a conservative – albeit not necessarily optimum – estimate of the required force may be conveniently obtained for mini-HDD applications:

(22)

$$F_{mini} = [w_b \, L_{bore} \, (1/3)] \, 1.6^n$$

WHERE

F_{mini} **= required pullback force, lbs**

L_{bore} = **horizontal pull length, ft**

w_b = **net upward normal force on pipe within borehole (Eq. 23), lb/ft**

n = **number of additional effective 90° route bends (Eq. 24)**

(23)

$$w_b = \left(\frac{1}{2}\right) OD^2 - w_p$$

for

OD = **pipe outside diameter, in.**

w_p = **weight of (empty) pipe, lb/ft**

Eq. 22 represents a simple formula, but which nonetheless provides a more realistic estimate than that obtained by the ASTM F1962 procedure (e.g., Eqs. 15), which is based on a well controlled boring operation, with minimal additional route bends and/or path corrections relative to that indicated in Figure 4. In comparison, Eq. 22, contains the parameter, n, intended to reflect the possibility of such additional effects in a mini-HDD application. A local distribution application may, for example, contain a deliberate (planned) bend required to follow the right-of-way. However, a more likely source of (unplanned) route curvature is that due to path corrections for such typically less well controlled operations. Thus,

(24)

$$n = n_1 + n_2$$

In Eq. 24, the term n_1 is equal to the number of deliberate/planned 90° route bends, and n_2 is equal to the cumulative curvature (equivalent number of 90° route bends) due to the unplanned path corrections. The latter will depend upon the local conditions and the experience and expertise of the operators, but a practical guideline is given in Eq. 25.

(25)

$$n_2 = \frac{L_{bore}}{500} \frac{2}{rod}$$

WHERE

rod = **diameter of drill rods, in.**

Thus, for 2-inch drill rods, the effective additional route curvature is one 90° route bend per 500 feet of bore length. For the hypothetical (extreme) path illustrated in Figure 9, the number of planned bends, n_1, is equal to 45°/90° (right turn) + 45°/90° (left turn), or ½ + ½ , which corresponds to one full 90° bend. The application of Eq. 25 results in additional (unplanned) effective route curvature, n_2, equal to 250/500, representing an additional half of a full (90°) bend. Thus, the total additional route curvature, beyond that required to descend and rise from the depth, is calculated as $n_1 + n_2 = 1 + ½ = 1½$.

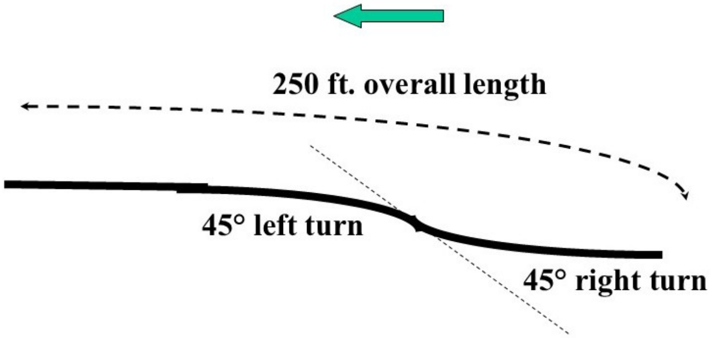

Figure 9 Additional Route Bends/Curvature

The pullback force of Eq. 22 is then compared to the appropriate (1-hour) safe pull force in Table 5 or 6.

In general, the use of Eqs. 15 (including ASTM F1962 or PPI-BoreAid) to estimate the required pull force for a mini-HDD operation may significantly underestimate the required load. Conversely, the use of Eqs. 22 – 25 to estimate the required pull force for a maxi-HDD operation may significantly overestimate the required load.

EXAMPLE CALCULATION An example calculation for selecting or confirming the DR for an HDD pipe, based on the installation load, is given in Appendix C.

TABLE 5

PE 4710 1-hour Safe Pull Strength* (1400 psi stress) vs. IPS Size

		Safe Pull Force, lbs				
Size	**Nom. OD**	**7**	**9**	**11**	**13.5**	**17**
1.25	1.660	1,484	1,197	1,002	831	671
1.5	1.900	1,944	1,568	1,312	1,089	879
2	2.375	3,038	2,450	2,050	1,702	1,373
3	3.500	6,597	5,321	4,453	3,695	2,983
4	4.500	10,910	8,796	7,361	6,109	4,931
6	6.625	23,640	19,070	15,950	13,240	10,690
8	8.625	40,060	32,310	27,040	22,440	18,110
10	10.750	62,240	50,200	42,010	34,860	28,140
12	12.750	87,550	70,620	59,090	49,040	39,580

*Table values are based on the minimum wall thickness of pipe

TABLE 6

PE 4710 1-hour Safe Pull Strength * (1400 psi stress) vs. DIPS Size

		Safe Pull Force, lbs				
Size	**Nom. OD**	**7**	**9**	**11**	**13.5**	**17**
3	3.96	8,445	6,812	5,700	4,731	3,818
4	4.80	12,410	10,010	8,375	6,950	5,610
6	6.90	25,640	20,680	17,310	14,360	11,590
8	9.05	44,110	35,580	29,770	24,710	19,940
10	11.10	66,360	53,520	44,790	37,170	30,000
12	13.20	93,840	75,690	63,330	52,560	42,430

*Table values are based on the minimum wall thickness of pipe

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Appendix A

Design Calculation Examples for Pipe Buckling/Collapse

Example 1

A 6-inch IPS DR 11 PE4710 pipe has been pulled under a railroad track, using mini-HDD, for use as an electrical conduit (non-pressure) application. The depth under the track is 10 ft. Determine the safety factor against long-term buckling.

Given Parameters

Nominal pipe size: OD = **6.625 in.**
Dimension Ratio: DR = **11**
Borehole depth: H_C = **10 ft**
Weight of soil: γ_S = **120 lb/ft^3**
E-80 live load: P_{SUR} = **1,100 lb/ft^2** (Table 3-5, Chapter 6)

PE Material Parameters

Wheel loading from a train will be applied for several minutes without relaxation. Repetitive trains crossing may accumulate. A conservative choice for the apparent elastic modulus for evaluating the ability to withstand the earth and (limited duration) dynamic loads is the 1000-hour modulus, while the 50-year modulus would be appropriate for evaluating the long-term resistance to the soil loads alone. See Table B.1.1 of Appendix B of Chapter 3.

E_{1000hr} = **46,000 psi**
E_{50yr} = **29,000 psi**
μ = Poisson's Ratio = 0.45

> **Note:** The apparent elastic moduli in Table B.1.1 of Chapter 3 apply at 73°F. Such values may be conservative, especially for extended periods, since the belowground temperature will be lower, with a correspondingly greater elastic modulus (see Table B.1.2 of Chapter 3). The examples in Appendix A conveniently use the conservative 73°F values.

Soil and Live Load Pressure on Pipe

Since this example considers a live (dynamic) load, it is assumed that the net imposed pressure, P_N equals the prism load of the earth plus the E-80 train pressure. Thus, Eq. 1 applies with the P_E pressure given by Eq. 7, plus the specified P_{SUR} pressure, but without the P_{GW} (groundwater pressure) and P_I (internal pressure) terms, which effects are not present, or:

$$\begin{aligned} P_N &= \frac{\gamma_S\, H_C}{144} + \frac{P_{SUR}}{144} \\ &= \mathbf{16.0\ psi} \end{aligned}$$

The ring deflection resulting from soil and live load pressures, assuming no side support, is given by Eq. 10, but with the P_E term replaced by $P_{N,}$ which includes the P_{SUR} load, or:

$$\frac{\Delta y}{OD} = 0.15\ (DR-1)^3\ P_N/E_{1000hr}$$
$$= 0.052\ = \mathbf{5.2\%}$$

Buckling Strength (Soil and Live Load Pressure)

Determine the corresponding unconstrained buckling pressure using Eq. 11, with:

μ = Poisson's Ratio = 0.45

f_o = 0.58 ovality compensation factor for 5.2% ovality from Figure 3 (assumes exceeds initial ovality, prior to loading)

$$P_{UC}\ = \frac{2\ E_{1000hr}}{(1-\mu^2)}\left(\frac{1}{DR-1}\right)^3 \frac{f_o}{N}$$
$$= \mathbf{66.9/N\ psi}$$

The safety factor, N, against buckling therefore equals 66.9/16.0, or **4.2**.

Long-term Soil Pressure on Pipe

For the present purposes, it will be conservatively assumed that the prism load is also applicable for the long-term soil pressure, or:

$$P_N\ = \frac{\gamma_S\ H_C}{144}$$
$$= \mathbf{8.3\ psi}$$

The corresponding ring deflection is then calculated as:

$$\frac{\Delta y}{OD} = 0.15\ (DR-1)^3\ P_N/E_{50yr}$$
$$= 0.043\ = \mathbf{4.3\%}$$

Table 2 indicates this long-term deflection is within acceptable limits.

Buckling Strength (Long-term Soil Pressure)

Determine the corresponding unconstrained buckling pressure using Eq. 11, with:

μ = Poisson's Ratio = 0.45

f_o = 0.67 ovality compensation factor for 4.3% ovality from Figure 3 (assumes exceeds initial ovality, prior to loading)

$$P_{UC}\ = \frac{2\ E_{50yr}}{(1-\mu^2)}\left(\frac{1}{DR-1}\right)^3 \frac{f_o}{N}$$
$$= \mathbf{48.7/N\ psi}$$

The safety factor, N, against buckling therefore equals 48.7/8.3, or **5.9**.

Note: If similar calculations are performed at 15 ft depth, then safety factor against buckling significantly exceeds the desired factor of 2.0. This example illustrates that pipe collapse is generally not anticipated for typical mini-HDD applications limited lo 15 ft depth.

Example 2

A 6-inch IPS DR 13.5 PE4710 pipe is being pulled under a small river, using mini-HDD, for use as an electrical conduit (non-pressure) application. The depth of the borehole beneath the ground surface is 15 feet. Assume the slurry weight is equal to 75 lb/ft^3. Calculate a) the maximum allowable (safe) pulling force and b) the safety factor against buckling during the installation. Assume that the pipe's initial ovality is 3% and that the maximum average (cross-sectional) tensile stress is 1200 psi. Assume an effective pulling time of 1 hour. The effective duration of the hydrokinetic pressure would be considerably less, with a correspondingly greater apparent elastic modulus to help withstand buckling, although is conservatively not considered in this example.

Given Parameters

Nominal pipe size: OD = **6.625 in.**
Dimension Ratio: DR = **13.5**
Maximum borehole depth: H_{MUD} = **15 ft**
Weight of slurry: γ_{MUD} = **75 lb/ft^3**
Initial ovality: **3%**
Maximum cross-sectional tensile stress: σ_{avg}= **1200 psi**

PE Material Parameters

Select the safe pull stress for PE4710 for a 1-hour pull duration from Table 1 and similarly select the apparent modulus for 1-hour duration from Table B.1.1 of Appendix B, Chapter 3.

σ_S **= 1400 psi**
E_{1hr} **= 78,000 psi**
μ **= Poisson's Ratio = 0.45**

Determine the safe pull force using Eq. 18:

$$F_S = \sigma_S \, \pi \, OD^2 \left(\frac{1}{DR} - \frac{1}{DR^2}\right)$$
$$= \mathbf{13,240\ lbs}$$

which value agrees with that in Table 5.

Slurry Pressure on Pipe

Equation 19 applies with the slurry pressure, P_{MUD}, given by Eq. 3, the hydrokinetic pressure, P_{HK}, assumed to be 10 psi, and without any internal pressure, P_I :

$$P_N = P_{MUD} + P_{HK} - P_I$$

$$= \frac{\gamma_{MUD}\, H_{MUD}}{144} + 10 \text{ psi}$$

$$= \mathbf{17.8\ psi}$$

Buckling Strength

Determine the critical buckling pressure during installation of the pipe, including the effect of the tension reduction factor of Eq. 21, based on the ratio:

$$r = \frac{\sigma_{avg}}{2\,\sigma_S}$$

$$= \mathbf{0.429}$$

for the assumed 1200 psi maximum stress and 1400 safe pull stress.

Thus, using Eq. 21:

$$f_r = \sqrt{[5.57 - (r + 1.09)^2]} - 1.09$$

$$= \mathbf{0.72}$$

Determine the corresponding unconstrained buckling pressure, under tension, using Eq. 20, with:

f_o **= 0.76 - ovality compensation factor (for 3% ovality).**

$$P_{tens} = \frac{2\,E_{1hr}}{(1-\mu^2)} \left(\frac{1}{DR-1}\right)^3 \frac{f_o\, f_r}{N}$$

$$= \mathbf{54.8\ psi}$$

The safety factor, N, against buckling during the pull therefore equals 54.8/19.4, or **2.8**.

Example 3

Determine the safety factor for long-term performance for the electrical conduit in Example 2. Assume the borehole is a maximum of 10 ft beneath the river bottom, and the depth of the river water is 3 ft. The groundwater surface is therefore 13 ft above the pipe, and there are 10 feet of riverbed deposits above the borehole, with an assumed saturated weight of 110 lb/ft^3.

Given Parameters

Nominal pipe size: OD = **6.625 in.**
Dimension Ratio: DR = **13.5**
Weight of water: γ_W **= 62.4 lb/ft^3**
Height water above pipe: H_W = **13 ft**
Weight of saturated riverbed soil: γ_{SAT} = **110 lb/ft^3**
Height soil cover: H_C = **10 ft**
Initial ovality: **3%**

PE Material Parameters

Select the long-term (50 year) apparent modulus for from Table B.1.1 of Appendix B, Chapter 3.

E_{50yr} = **29,000 psi**
μ = **Poisson's Ratio = 0.45**

Soil and Water Pressure on Pipe

The (asymmetric) ring deformation of the pipe depends on the pressure, P_E , exerted by the buoyant weight of the saturated soil, which is obtained from Eq. 9 by eliminating the (symmetric) effect of the water pressure, P_{GW}. Thus,

$$\begin{aligned} P_E &= \frac{\gamma_B\, H_C}{144} \\ &= \frac{(\gamma_{SAT} - \gamma_W)\, H_C}{144} \\ &= \mathbf{3.3\ psi} \end{aligned}$$

The resulting ring deflection, assuming no side support, due to the soil load is determined using Eq. 10.

$$\begin{aligned} \frac{\Delta y}{OD} &= 0.15\,(DR - 1)^3\, P_E/E_{50yr} \\ &= 0.033 \quad = \mathbf{3.3}\% \end{aligned}$$

This induced deflection (ovality) may potentially increase, or decrease, that initially present. It would be excessively conservative to assume that this additional deflection is directly additive to the assumed initial ovality (3%). A reasonable engineering assumption, as adopted in PPI-BoreAid, would be to use the larger of the initial deflection (3%) and that due to earth pressure deflection (3.3%); i.e., 3.3% in this case.

To avoid buckling, the pipe must withstand the total pressure on the pipe, including the buoyant weight of the saturated soil, plus the hydrostatic pressure of the water. Using Eq.9:

$$\begin{aligned} P_E + P_{GW} &= \frac{\gamma_B H_C + \gamma_W H_W}{144} \\ &= \mathbf{8.9\ psi} \end{aligned}$$

Buckling Strength

Determine the corresponding unconstrained buckling pressure using Eq. 11, with:

μ = Poisson's Ratio = 0.45
f_o = 0.74 ovality compensation factor for 3.3% from Figure 3

$$P_{UC} = \frac{2\,E_{50yr}}{(1-\mu^2)}\left(\frac{1}{DR-1}\right)^3 \frac{f_o}{N}$$
$$= \mathbf{27.6\ psi}$$

The safety factor, N, against buckling therefore equals 27.6/8.9, or **3.1**.

Slurry Pressure on Pipe

It is also reasonable to consider the possibility that the slurry will remain in essentially a fluidic state throughout the life of the pipe. In this case, Eq. 2 applies (without any internal pressure, P_I , with the slurry pressure, P_{MUD}, given by Eq. 3, and the slurry weight of 75 lb/ft^3 (Example 2). Thus,

$$P_N = P_{MUD} - P_I$$
$$= \frac{\gamma_{MUD}\,H_{MUD}}{144}$$
$$= \mathbf{7.8\ psi}$$

Buckling Strength

Determine the corresponding unconstrained buckling pressure using Eq. 11, with:

μ = Poisson's Ratio = 0.45
f_o = 0.76 - ovality compensation factor (for initial 3% ovality).

$$P_{UC} = \frac{2\,E_{50yr}}{(1-\mu^2)}\left(\frac{1}{DR-1}\right)^3 \frac{f_o}{N}$$
$$= \mathbf{28.3\ psi}$$

The safety factor, N, against buckling therefore equals 28.3/7.8, or **3.6**, which exceeds the safety factor for the previous assumed condition of the pipe subject to the buoyant weight of the saturated soil plus the groundwater.

> **Note:** The 6 inch pipe of Examples 2 and 3 pipe was installed using mini-HDD, which is typically limited to the present assumed depth of 15 ft. Pipe collapse or buckling is generally not anticipated for such applications, as demonstrated by these examples.

APPENDIX B (MAXI-HDD: ASTM F1962 & PPI-BOREAID™)

Design Calculation Example for Loads During Installation

ASTM F1962

For the river crossing and HDD geometry of Figure 4, find the estimated force required to pull back a 24-inch IPS DR 11 PE4710 pipe using the general methodology of ASTM F1962 for maxi-HDD, as outlined in this chapter. Also, determine the safety factor against collapse during the installation, assuming the pipe's initial ovality is 3%. Assume the PE pipe is placed at a depth of 35 ft within a crossing 870 ft long with a 10 degree entry angle and a 15 degree exit angle. It is recognized that the actual pullback force will vary, depending on backreamer size and selection; drilling fluid; borehole stability; soil conditions; driller expertise; and other application circumstances.

Given Parameters

Nominal pipe size: OD = **24 in.**
Dimension Ratio: DR = **11**
Minimum thickness: t = OD/DR = **2.18 in**
Maximum borehole depth: H_{MUD} = **35 ft**
Weight of water: γ_W = **62.4 lb/ft^3**
Initial ovality: **3%**

PE Material Parameters

Select the safe pull stress for PE4710 for 12-hour pull duration from Table 1 and similarly select the apparent elastic modulus for 12 hours duration from Table B.1.1 of Appendix B, Chapter 3. The effective duration of the hydrokinetic pressure would be considerably less, with a correspondingly greater apparent elastic modulus to help withstand buckling, although is conservatively not considered in this example.

σ_S = **1330 psi**
E_{12hr} = **63,000 psi**
μ = **Poisson's Ratio = 0.45**

Path Geometry (Figure 4; see ASTM F1962)

Depth of bore: H = **35 ft**
Pipe entry angle: α = **10 deg (0.175 radians)**
Pipe exit angle: β = **15 deg (0.262 radians)**
Bore length: L_{bore} = **870 ft**
Excess length: L_1 = **100 ft (excess length of pipe being pulled, remaining at end of pull; see ASTM F1962)**

Average radius of curvature for path A-B at pipe entry (α in radians)

$$R_{entry} = 2\,H/\alpha^2 = \mathbf{2{,}286\ ft}$$

Average radius of curvature for path C-D at pipe exit (β in radians)

$$R_{exit} = 2\,H/\beta^2 = \mathbf{1{,}020\ ft}$$

For the 24-inch pipe, these radii are an order of magnitude greater than the minimum recommended to minimize the effect of ovaling. It is assumed that the local bending radii at the entry and exit points to the borehole are sufficiently controlled as to not cause any permanent deformation.

Horizontal distance required to achieve depth or rise to the surface at pipe entry (A) or exit (D)

$L_2 = 2H/\alpha$
$= \mathbf{400\ ft}$

$L_4 = 2H/\beta$
$= \mathbf{267\ ft}$

$L_3 = L_{bore} - L_2 - L_3$
$= \mathbf{203\ ft}$

Estimated Pulling Force

Weight of empty pipe

$SG_{HDPE} = 0.95$ (specific gravity PE4710)

$w_p = \pi\ OD^2\ [(DR\text{-}1)/DR^2]\ \gamma_W\ SG_{HDPE} \times (1\ ft/12\ in)^2$
$= \mathbf{61.6\ lb/ft}$

Net upward buoyant force on empty pipe surrounded by mud/slurry

$SG_{slurry} = 1.5$ (specific gravity mud/slurry; see ASTM F1962)

$w_{buoyant} = \pi\ (OD^2/4)\ \gamma_W\ SG_{slurry} \times (1\ ft/12\ in)^2$ = upward buoyant force
$= \mathbf{294.0\ lb/ft}$

$w_b = w_{buoyant} - w_p$
$= \mathbf{232.4\ lb/ft}$ = net upward buoyant force

Coefficients of friction

$v_g = \mathbf{0.40}$ (coefficient of friction external to borehole; ASTM F1962 suggests 0.1 or 0.5)
$v_b = \mathbf{0.25}$ (coefficient of friction internal to borehole; ASTM F1962 suggests 0.3)

Pulling force (Eqs. 15)

$$F_A = \exp(v_g\ \alpha)\left[v_g\ w_p\ (L_1 + L_2 + L_3 + L_4)\right]$$
$$= \mathbf{25,620\ lbs}$$

$$F_B = \exp(v_b\ \alpha)\left[F_1 + v_b\ w_b\ L_2 + w_b\ H - v_g\ w_p\ L_2 \exp(v_g\ \alpha)\right]$$
$$= \mathbf{48,520\ lbs}$$

$$F_C = F_2 + v_b\ w_b\ L_3 - \exp(v_b\ \alpha)\left[v_g\ w_p\ L_3 \exp(v_g\ \alpha)\right]$$
$$= \mathbf{54,720\ lbs}$$

$$F_D = \exp(v_b\ \beta)\left\{F_3 + v_b\ w_b\ L_4 - w_b\ H - \exp(v_b\ \alpha)\left[v_g\ w_p\ L_4 \exp(v_g\ \alpha)\right]\right\}$$
$$= \mathbf{58,430\ lbs}$$

Hydrokinetic (fluidic) drag (Eq. 16)

P_{HK} = **10 psi** (see ASTM F1962

D_h = 1.5 OD (see ASTM F1962)

= **36 in.**

$$\Delta_F = P_{HK}\frac{\pi}{8}\left(D_h^{\,2} - OD^2\right)$$

$$= \mathbf{2,830\ lbs}$$

Maximum tensile stress (Eqs. 17)

$F_T = F_D + \Delta_F$

= **61,260 lbs**

$R = R_{exit}$

= 1020 ft = **12,237 in**

$$\sigma_{avg} = \frac{F_T}{\pi\, t\, (OD - t)} = \mathbf{410\ psi}$$

$$\sigma_{bend} = \frac{E_{12hr} \times OD}{2\,R} = \mathbf{62\ psi}$$

$$\sigma_T = \sigma_{avg} + \sigma_{bend} = \mathbf{472\ psi}$$

This stress value is much lower than the 1,330 psi safe pull stress, even when considering the additional bending stresses along the exit segment C-D.

Breakaway link

In general, breakaway links should be set so that pullback force applied to pipe does not exceed an average (cross-section) stress equal to the safe pull stress. The breakaway force, F_{break} should therefore be set as:

$F_{brkwy} = \sigma_S$ x cross-section area

where:

cross-section area = $(\pi/4)\ (OD^2 - ID^2)$

ID (inner diameter) = OD – 2t

t = OD/DR

Thus,

cross-section area = $\pi\ t\ (OD - t)$

or

= $\pi\ OD^2\ [1/DR - 1/DR^2]$

= **149.6 in²**

F_{brkwy} = **198,900 lbs**

which agrees with the value in Table 3 for a 24-inch IPS DR 11 PE4710 pipe. In practice, a rounded-down value, such as 5% to 10% lower, may be conveniently selected to account for bending stress; e.g., 180,0000 - 190,000 lbs.

Hydrostatic Pressure

External static head pressure (Eqs. 3 and 19)

$\gamma_{MUD} = 1.5\ \gamma_W$

$= \mathbf{93.6\ lb/ft^3}$

$P_{HK} = \mathbf{10\ psi}$

$$P_N = P_{MUD} + P_{HK} - P_I$$

$$= \frac{\gamma_{MUD}\ H_{MUD}}{144} + 10\ \text{psi}$$

$$= \mathbf{32.75\ psi}$$

Buckling Strength (Eqs. 20 and 21)

f_o = **0.76** ovality compensation factor, 3% ovality

$$r = \frac{\sigma_{avg}}{2\ \sigma_S}$$

$$= \mathbf{0.154}$$

$$f_r = \sqrt{[5.57 - (r + 1.09)^2]} - 1.09$$

$$= \mathbf{0.92}$$

$$P_{tens} = \frac{2\ E_{12hr}}{(1 - \mu^2)} \left(\frac{1}{DR - 1}\right)^3 \frac{f_o\ f_r}{N}$$

$$= \mathbf{110\ psi}$$

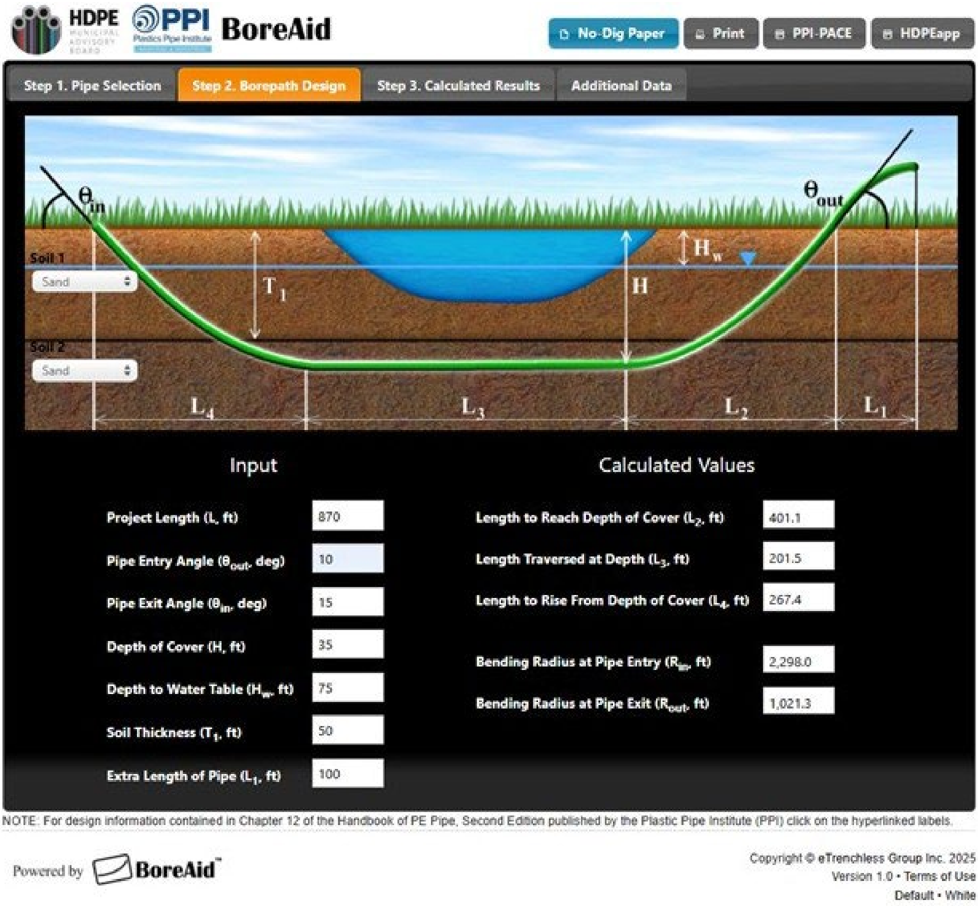
Safety factor against collapse

$N = 110/32.75$

$= \mathbf{3.4}$

PPI-BOREAID™

The same installation will now be considered using PPI-BoreAid. PPI-BoreAid also follows the general methodology of ASTM F1962, including its suggested values for the coefficients of friction. However, in the calculations above, somewhat lower values are considered for the coefficients of friction, as part of the investigation, as may be deemed appropriate by the engineer.



Input

Project Length (L, ft) 870
Pipe Entry Angle (θ_{out}, deg) 10
Pipe Exit Angle (θ_{in}, deg) 15
Depth of Cover (H, ft) 35
Depth to Water Table (H_w, ft) 75
Soil Thickness (T_1, ft) 50
Extra Length of Pipe (L_1, ft) 100

Calculated Values

Length to Reach Depth of Cover (L_2, ft) 401.1
Length Traversed at Depth (L_3, ft) 201.5
Length to Rise From Depth of Cover (L_4, ft) 267.4
Bending Radius at Pipe Entry (R_{in}, ft) 2,298.0
Bending Radius at Pipe Exit (R_{out}, ft) 1,021.3

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Figure B.1 PPI-BoreAid: Specified Bore Path

Figure B.1 shows the values for the specified geometry inserted into PPI-BoreAid. The calculated values for the various distances, as well as the bend radii, are shown. These values are in agreement with those calculated above, except for small differences due to roundoff.

The results are shown in Figures B.2 and B.3, and indicate that this relatively short, 870 ft, maxi-HDD installation is well within the safe or allowable pulling strength of the considered (DR 11) pipe, even without considering load reductions using external roller supports or internal (water) ballast. This is also the case if a thinner-walled (DR 13.5) pipe is selected. Similarly, the collapse strengths of the DR 11 pipe during the installation or operational phases have safety factors significantly in excess of double the corresponding applied pressures, as desired. The DR 13.5 pipe would also be acceptable.

● Show Calculated Values
○ Show Factor of Safety

*** Many design and material parameters are assumed in these calculations based upon suggested values from ASTM F1962 - Click here for a complete list of assumed parameter values.

Applied Loading: Earth pressure = 5.1 psi. Water pressure = 0.0 psi. Mud pressure = 22.7 psi.

	HDPE-PE4710		OPERATIONAL		INSTALLATION				
	IPS Nom. OD inches	IPS DR	Deflection % OD	Critical Collapse - Full - 50 yr (pressure pipe) psi	Critical Collapse - 1 hr psi	Critical Collapse - 10 hr psi	Pull Back Force lbs	Allowable Pullback lbs	Status
No Rollers & No Ballast	24	9	1.0	109	275	229	68,638	237,700	PASS
	24	11	1.9	56	139	116	71,414	198,902	PASS
	24	13.5	3.7	27	69	58	73,835	165,069	PASS
With Rollers & No Ballast	24	9	1.0	109	276	230	64,883	237,700	PASS
	24	11	1.9	56	139	116	68,272	198,902	PASS
	24	13.5	3.7	27	70	58	71,228	165,069	PASS
With Rollers & Ballast	24	9	1.0	109	285	237	30,341	237,700	PASS
	24	11	1.9	56	145	121	30,049	198,902	PASS
	24	13.5	3.7	27	74	62	29,794	165,069	PASS

Red highlighted cells indicate that calculated values may be less than recommended values.

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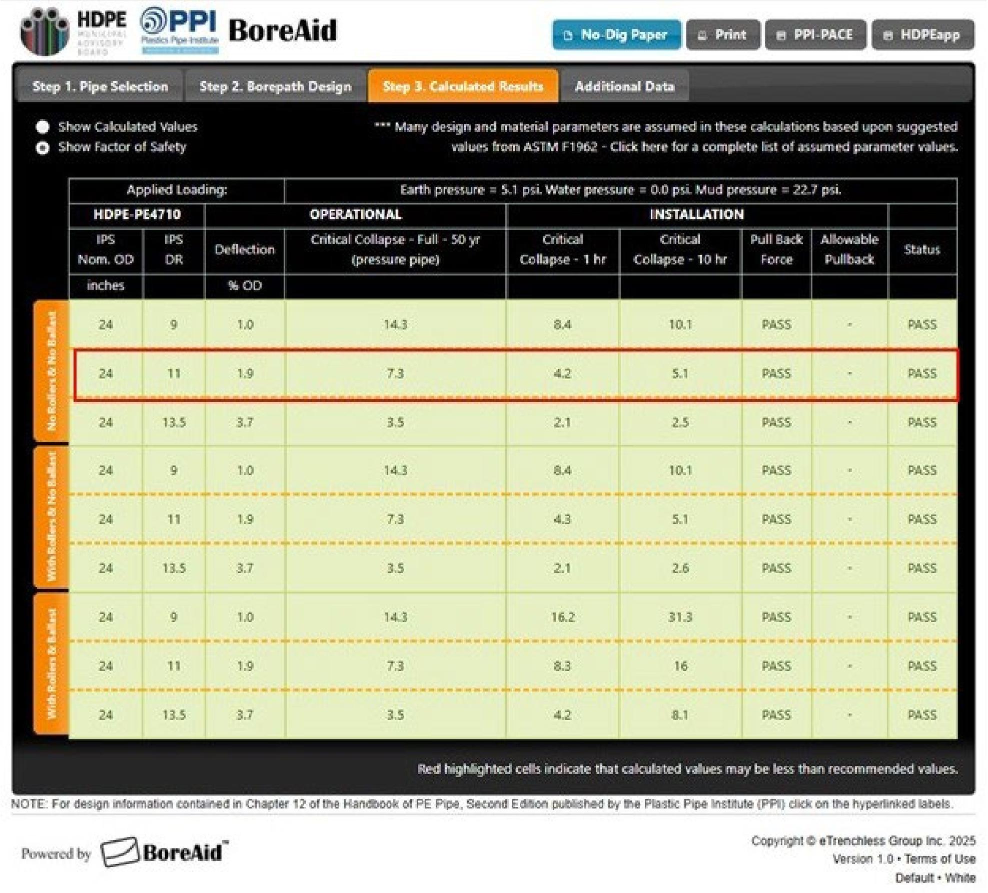
Figure B.2 PPI-BoreAid: Results (Pipe Strengths vs. Effects of Loads)

There is, however, a quantitative difference between the PPI-BoreAid pullback force and that calculated above. Figure B.2 shows a required pull back force of 71,414 lbs, which is significantly greater than the 61,260 lbs calculated above. This latter estimate is based on a borehole coefficient of friction of 0.25 in comparison to the 0.3 value in PPI-BoreAid (and ASTM F1962). It is noted that the allowable pullback force (198,902 lbs) shown in Figure B.2 is in agreement with that calculated above for the breakaway link, or as provided in Table 3.

PPI-BoreAid provides several collapse strengths, including the 50-year strength, and two installation strengths – both a 1-hour (139 psi) and a 10-hour (116 psi) value. The 1-hour effective strength withstands the combination of the hydrostatic pressure of the slurry plus the hydrokinetic pressure (10 psi) only present during operation of the backreamer. The 10-hour effective strength only withstands the hydrostatic pressure of the slurry. In comparison to the 116 psi (for the assumed 10-hour installation period), the somewhat more conservative calculation above gives 110 psi (for the assumed 12-hour installation period), due to its slightly lower effective elastic modulus and the use of the maximum tension to determine a slightly lower tension factor, f_r, as discussed previously.

The safety factors in Figure B.3 indicate a value of 5.1 for the 10-hour period. In comparison, the procedure above provides a safety factor of 3.4. The difference is primarily due to the latter conservatively including the 10 psi hydrokinetic pressure, albeit not present for the full installation period.



○ Show Calculated Values
● Show Factor of Safety

*** Many design and material parameters are assumed in these calculations based upon suggested values from ASTM F1962 - Click here for a complete list of assumed parameter values.

	Applied Loading:			Earth pressure = 5.1 psi. Water pressure = 0.0 psi. Mud pressure = 22.7 psi.					
	HDPE-PE4710		OPERATIONAL		INSTALLATION				
	IPS Nom. OD	IPS DR	Deflection	Critical Collapse - Full - 50 yr (pressure pipe)	Critical Collapse - 1 hr	Critical Collapse - 10 hr	Pull Back Force	Allowable Pullback	Status
	inches		% OD						
No Rollers & No Ballast	24	9	1.0	14.3	8.4	10.1	PASS	-	PASS
	24	11	1.9	7.3	4.2	5.1	PASS	-	PASS
	24	13.5	3.7	3.5	2.1	2.5	PASS	-	PASS
With Rollers & No Ballast	24	9	1.0	14.3	8.4	10.1	PASS	-	PASS
	24	11	1.9	7.3	4.3	5.1	PASS	-	PASS
	24	13.5	3.7	3.5	2.1	2.6	PASS	-	PASS
With Rollers & Ballast	24	9	1.0	14.3	16.2	31.3	PASS	-	PASS
	24	11	1.9	7.3	8.3	16	PASS	-	PASS
	24	13.5	3.7	3.5	4.2	8.1	PASS	-	PASS

Red highlighted cells indicate that calculated values may be less than recommended values.

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Figure B.3 PPI-BoreAid: Results (Factors of Safety)

In summary, while it is recognized that the detailed calculations above provide flexibility for the engineer to consider the effects of variations in some of the parameters, from a practical perspective, PPI-BoreAid is an essential tool in performing initial studies. PPI-BoreAid incorporates reasonable engineering judgment in determining and utilizing effective durations of various loads. Although not a consideration in this example, PPI-Bore-Aid also performs the considerably more complicated calculations to determine the effects of the soil and groundwater loads on the pipe, and its long-term stability.

APPENDIX C (MINI-HDD: MAB-7)

Design Calculation Example for Loads During Installation

A 6-inch IPS DR 11 PE4710 pipe is being pulled under an obstacle, through a nominally straight 590 ft borehole, using mini-HDD with 2-inch drill rods. Using the methodology of MAB-7, calculate a) the maximum allowable (safe) pulling force and b) the estimated pull force. It is recognized that the actual pullback force will vary, depending on local conditions; driller expertise; and other application circumstances.

Given Parameters

Nominal pipe size: OD = **6.625 in.**
Dimension Ratio: DR = **11**
Borehole length: L_{bore} = **590 ft**
Planned route bends: n_1 = **0**
Drill rod size: rod = **2 in.**

PE Material Parameters

Select the safe pull stress for PE4710 for 1-hour pull duration, from Table 1.

σ_S = **1400 psi**

Determine the safe pull force using Eq. 18:

$$\begin{aligned} F_S &= \sigma_S\, \pi\, OD^2 \left(\frac{1}{DR} - \frac{1}{DR^2}\right) \\ &= \mathbf{15,950\ lbs} \end{aligned}$$

which value agrees with that in Table 5.

Estimated Pulling Force (MAB-7)

Weight of empty pipe (from manufacturer, or as calculated below)

SG_{HDPE}= 0.95 (specific gravity PE4710)

$$\begin{aligned} w_p &= \pi\, OD^2\, [(DR-1)/DR^2]\, \gamma_W\, SG_{HDPE} \text{ x } (\text{ft}/12 \text{ in})^2 \\ &= \mathbf{4.69\ lb/ft} \end{aligned}$$

Net upward buoyant force on pipe (Eq. 23)

$$\begin{aligned} w_b &= \left(\frac{1}{2}\right) OD^2 - w_p \\ &= \mathbf{17.3\ lb/ft} \end{aligned}$$

Additional route bends (Eq. 25)

$$\begin{aligned} n_2 &= \frac{L_{bore}}{500}\, \frac{2}{rod} \\ &= \mathbf{1.18} \end{aligned}$$

Total additional bends route bends (Eq. 24)

$$\begin{aligned} n &= n_1 + n_2 \\ &= \mathbf{1.18} \end{aligned}$$

Pull force (Eq. 22)

$$F_{mini} = [w_b\, L_{bore}\, (1/3)]\, 1.6^n$$
$$= 3{,}402 \text{ lbs x } \mathbf{1.74}$$
$$= \mathbf{5{,}920\ lbs}$$

This force is much lower than the 15,950 lbs safe pull force.

> **Note:** This example addressed an actual field installation in which the recorded peak pulling tension was 5,620 lbs. This extremely close agreement (within 5%) is not normally to be expected, due to the many variables. In some cases, the results may be overly conservative, or, conversely, may possibly underestimate the actual peak pulling force. Nonetheless, the results demonstrate the importance of accounting for the additional (unplanned) route bends, which effect almost doubles the required pulling force (amplification factor of 1.74).